Lecture Notes in Civil Engineering

SURVEYING

Chapters

1. Fundamental Definitions & Concepts
2. Horizontal Distance Measurement & Chain Surveying
3. Plane Table Surveying
4. Levelling
5. Study of Theodolite
6. Theodolite Traverse
7. Trigonometric Levelling
8. Tacheometry

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CHAPTER 1

FUNDAMENTAL DEFINITIONS & CONCEPTS

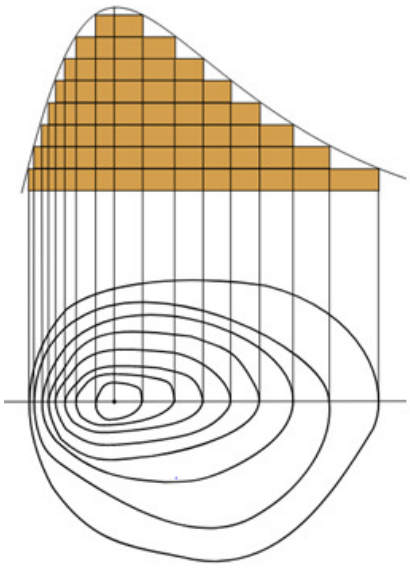
Surveying is the art of determining the relative position of points on the surface of earth. Here, the positions are established on a horizontal plane.

Levelling is the art of determining the relative heights or elevations of different points on earth's surface. Here, the positions are established on a vertical plane.

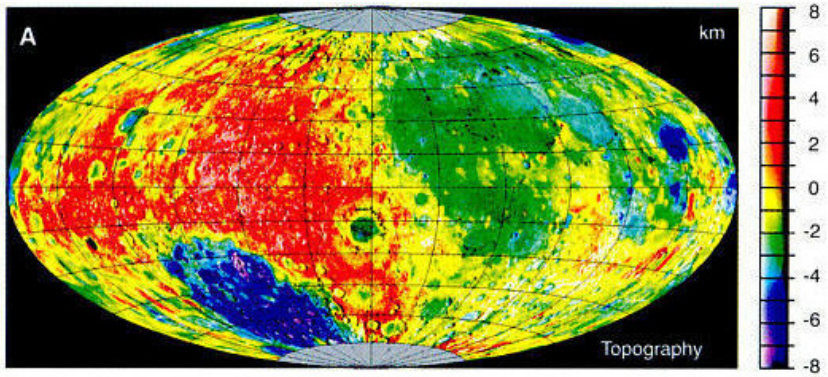
OBJECTIVE OF SURVEY

Primary object of survey is the preparation of **plan** or **map.**

- Plan/Map is the representation of a space (ground, objects etc), as projected on a horizontal plane, but drawn to scale..
  - If the scale is small => it is called Map
  - If the scale is large => it is called Plan
- Elevation is represented on these by means of contour lines or shading.



Contour Lines



Shaded Topographic Map

PRIMARY DIVISIONS OF SURVEY

(a) Plane Surveying

- Considers earth's surface as a plane; curvature of earth is neglected
- Line joining any two points is considered as straight line; angles are considered as plane angles; triangle formed by joining survey lines is considered as Plane Triangle
- Requires the knowledge of plane trigonometry and geometry
- All level lines are considered as straight & plumb lines are considered parallel
- Applicable only to smaller areas (< 250 km2)
- Degree of accuracy is low
- Generally employed to survey vast extend of areas without much accuracy.

(b) Geodetic Surveying (or Trigonometrical Surveying)

- Considers the curvature of earth
- Line joining any two points is an arc; angles are considered as spherical angles; the figure formed by joining 3 points is a Spherical Triangle
- Requires the knowledge of Spherical trigonometry
- Applied to large areas (> 250 km2)
- Degree of accuracy is high. Therefore, requires refined instruments and precise adjustments & observations
- Hence useful to establish control points at large distance from on another on the surface of earth precisely.

CLASSIFICATION OF SURVEYS

Surveys are classified into:

(a) Based on nature of field

1. **Land Surveys** – subdivided into:
   i) *Topographical Surveys* – to determine the natural features (such as hills, valleys, rivers, etc.) and artificial features (roads, rails, canals, etc.) of the country.
   ii) *Cadastral Surveys* – to fix the boundaries and area of land. Used if land property is transferred from one owner to another.
   iii) *City Surveys* – used for construction of streets, water supply systems, sewers etc.
2. **Marine / Hydrographic Surveys** – survey of waterbodies for purpose of navigation, water supply, harbor works, and for the determination of Mean Sea Level.
3. **Astronomical surveys** – determining the absolute location of any point on the earth's surface with respect to any fixed reference such as sun or star.

(b) Based on object of survey

1. **Engineering Surveys** – to collect data and determine quantities for construction of any engineering work, such as buildings, roads, bridges, reservoirs, etc. They are subdivided into:
   i) *Reconnaissance Surveys* – to obtain brief rough data to ascertain the feasibility of work and to find its rough cost.
   ii) *Preliminary Surveys* – to collect precise data to choose best location for work and to estimate exact quantites and prepare detailed estimates.
   iii) *Location Surveys* – to set out the work on the ground.
2. **Military Surveys** – to determine points of strategic importance
3. **Mine Surveys** – to explore mineral resources
4. **Geological Surveys** – to explore earth's crust
5. **Archeological Surveys** – used to unearth objects of archeological importance

(c) Based on instruments used

1. Chain surveys
2. Theodolite Surveys
3. Compass Surveys
4. Tacheometric Surveys

5. Plane Table Surveys
6. Total Station Surveys
7. Photogrammetric surveys
8. Global Positioning System

PRINCIPLES OF SURVEYING

Fundamental principles of Plane Surveying are:

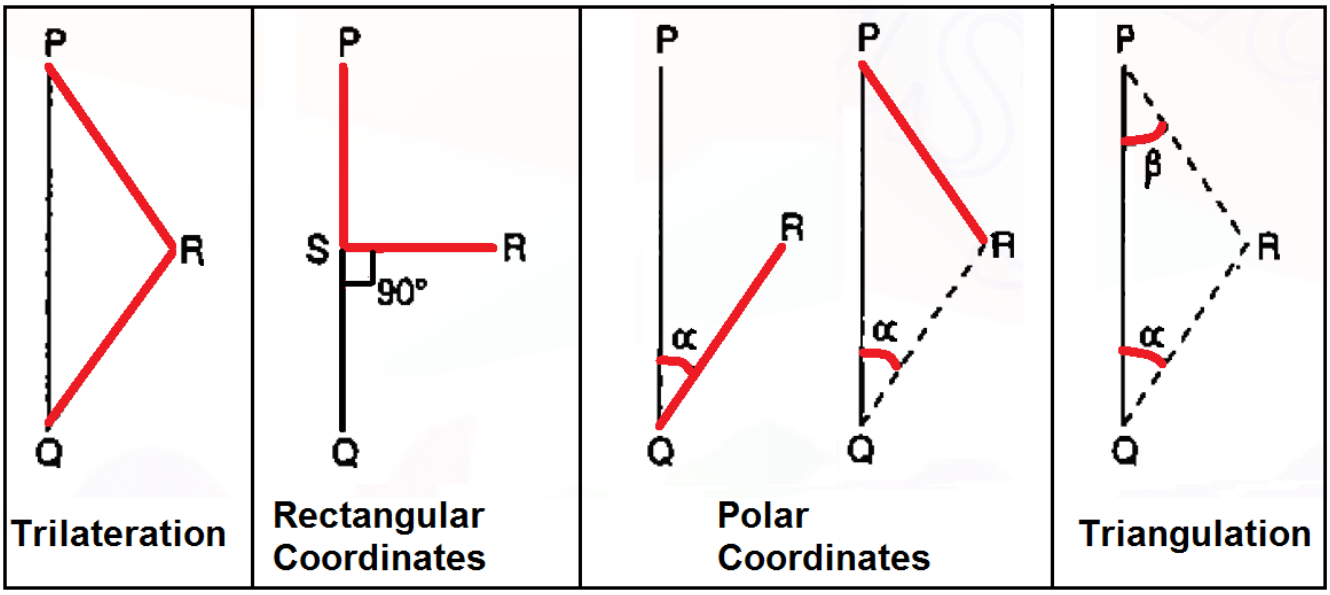
(a) To work from the whole to the part

- As the first step in surveying, the area to be surveyed is divided into **large triangles** which are surveyed with **high accuracy**. They are further subdivided into **small triangles** which are surveyed with **lesser accuracy**. This is known as *working from the whole to the part*.
- The aim of this principle is:
  - It prevents error accumulation
  - It limits and localizes minor errors
- But, on the other hand, *working from the part to the whole* results in magnification of errors and must be avoided.

(b) To fix the position of new stations by atleast two independent processes

If points P & Q are two existing points, a new point R is established with the help of two independent measurements, such as:

i) Trilateration: Distances PR and QR are measured
ii) Rectangular coordinates: Distances PS and SR are measured
iii) Polar coordinates: Angle PQR and distance QR are measured
iv) Triangulation: Angles PQR and QPR are measured



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TYPES OF MEASUREMENTS IN SURVEYING

(1) Linear Measurements – this includes:

i) Horizontal Distances
ii) Vertical Distances

(2) Angular Measurements – this includes:

i) Horizontal Angles
ii) Vertical Angles

UNITS OF MEASUREMENT

- In India, every unit of weight or measures shall be based on **Metric System**, as per **Standards Of Weights And Measures Act (1976)**. The international system of units as recommended by the General Conference on Weights and Measures, and such additional units as may be recommended by the International Organisation of Legal Metrology, shall be the units of the metric system.
- According to this,
  - the unit for linear measurements is **metre (m)**
  - the unit for angular measurements is **radian (rad)**
- Some of the other units used and its conversions are:
  - **Length:**
    - 12 inches = 1 feet
    - 3 feet = 1 yard
    - 6 feet = 1 fathom
    - 16.50 feet = 1 rod = 1 pole = 1 perch
    - 66 feet = 1 chain
    - 10 chain = 1 furlong
    - 1 mile = 1.610 kilometres = 5280 feet
    - 1 nautical mile = 1.852 kilometres = 6076.10 feet (nominal length of a minute of latitude, or of longitude at the equator)
  - **Area:**
    - 1 cent = $40.47 m^2$
    - 1 acre = 100 cents
    - 1 are = $100 m^2$
    - 1 hectare = $10000 m^2$
  - **Angle:**
    - 1 degree = $\frac{\pi}{180}$ radian
    - 1 minute = $\frac{1}{60}$ degree
    - 1 second = $\frac{1}{60}$ minute

CHAPTER 2

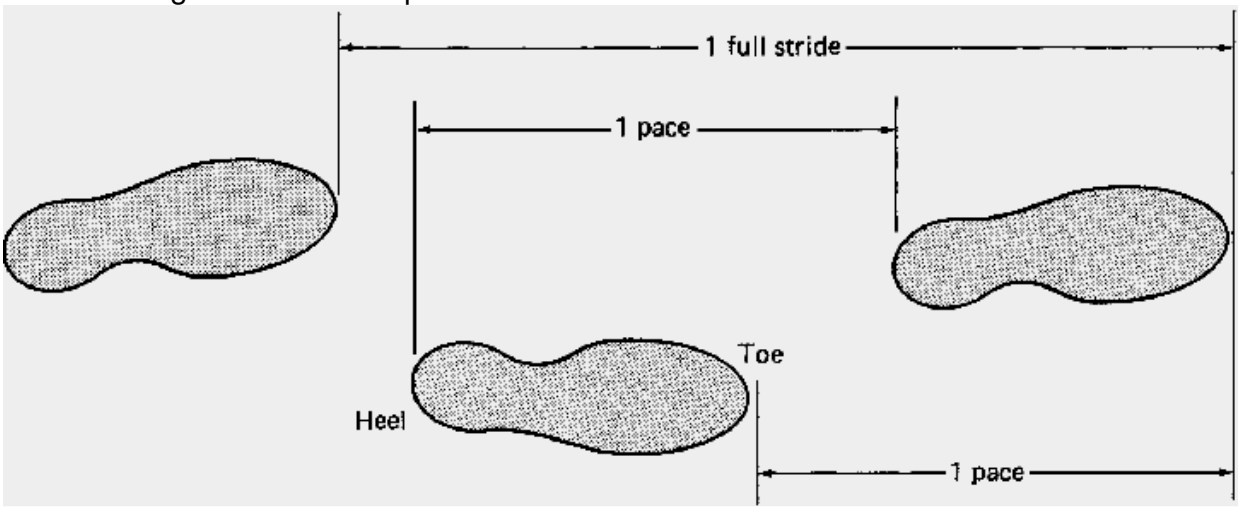
HORIZONTAL DISTANCE MEASUREMENT & CHAIN SURVEYING

METHODS OF HORIZONTAL DISTANCE MEASUREMENT

In surveying, linear measurements have been obtained by many different methods. These include (1) pacing, (2) odometer readings, (3) optical rangefinders, (4) tacheometry, (5) subtense bars, (6) chain, (7) tape, (8) electronic distance measurement (EDM).

1) Pacing

- Distances obtained by pacing are sufficiently accurate for many purposes in surveying, engineering, geology, agriculture, forestry, and military field sketching.
- Pacing is also used to detect blunders that may occur in making distance observations by more accurate methods.
- Pacing consists of counting the number of steps (or paces), in a required distance. For this, the length of an individual's pace must be determined first. A unit pace is taken as the distance between the left and right toes (or left and right heels) or vice versa of two successive positions. It is expressed in terms of meters per pace. This is best done by walking with natural steps back and forth over a level course of known distance (at least 100m long), and dividing the known distance by the average number of steps.

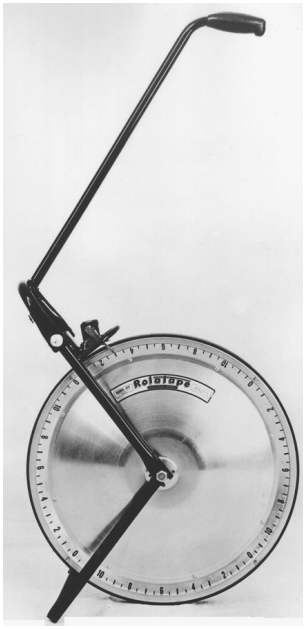


- Then the individual shall walk naturally along the line to be measured and count his paces. Length of the given line is given by:

Distance = Unit pace x Number of paces

- For long distances, a pocket instrument called a ***pedometer*** can be carried to register the number of paces, or a ***passometer*** attached to the body or leg counts the steps. Some surveyors prefer to count ***strides***, a stride being two paces.
- Distances can be measured with an accuracy of about 1:100.

2) Odometer



- A simple *measuring wheel* mounted on a rod can be used to determine distance by pushing the rod and rolling the wheel along the line to be measured.
- An attached device called an *odometer* serves to count the number of turns, or revolutions, of the wheel. From the known circumference of the wheel and the number of revolutions, distances can be determined.
- Distances can be measured with an accuracy of about 1:200.
- This device is particularly useful for rough measurements of distance along curved lines, as well as for quick checks of route survey measurements or construction pay quantity measurements.

3) Optical Range Finders

- These instruments operate on the same principle laid down by the fundamental lens formula,

$$\frac{1}{f_1} + \frac{1}{f_2} = \frac{1}{f}$$

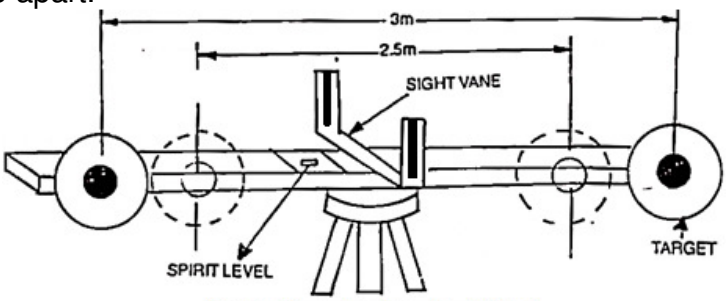
  where f_1 is the distance from the lens to the image formed in the plane of crosshairs, f_2 the distance from the lens to the object, and f the lens *focal length.*
- Basically, when the object is perfectly focused, the object distance f_2 in the above equation can be obtained since the focal length f and image distance f_1 are known.
- These instruments are capable of accuracies of 1:50 at distances up to 50m, but accuracy diminishes as the length increases.
- They are suitable for reconnaissance, sketching, or checking more accurate observations for mistakes.

4) Tacheometry

- Tacheometry is a surveying method used to quickly determine the horizontal distance and level difference between two points.
- It uses a Theodolite with a stadia diaphragm (known as Tacheometer) and leveling staff, details of which are given separately as a chapter later.
- An accuracy of 1:500 can be achieved.

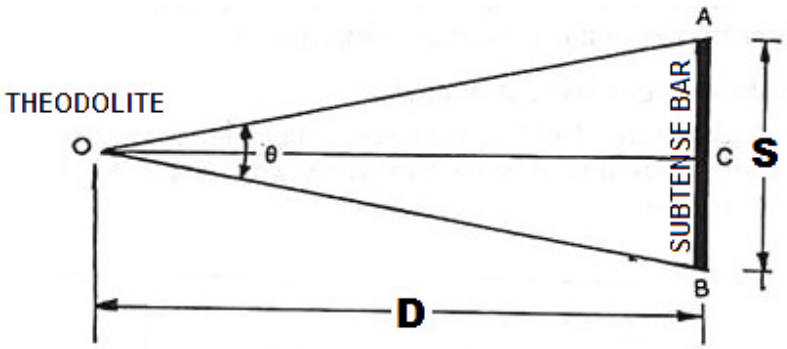
5) Subtense Bar

- This indirect distance-measuring procedure involves using a theodolite to read the horizontal angle subtended by two targets precisely spaced at a fixed distance apart on a subtense bar. A subtense bar is a horizontal staff with targets fixed at a known distance apart.



SUBTENSE BAR

- The bar is mounted on a tripod and is placed at right angles to the line of sight for making observations.



- The unknown distance is computed from the known target spacing (**S**) and the measured horizontal angle (**Θ**), as:

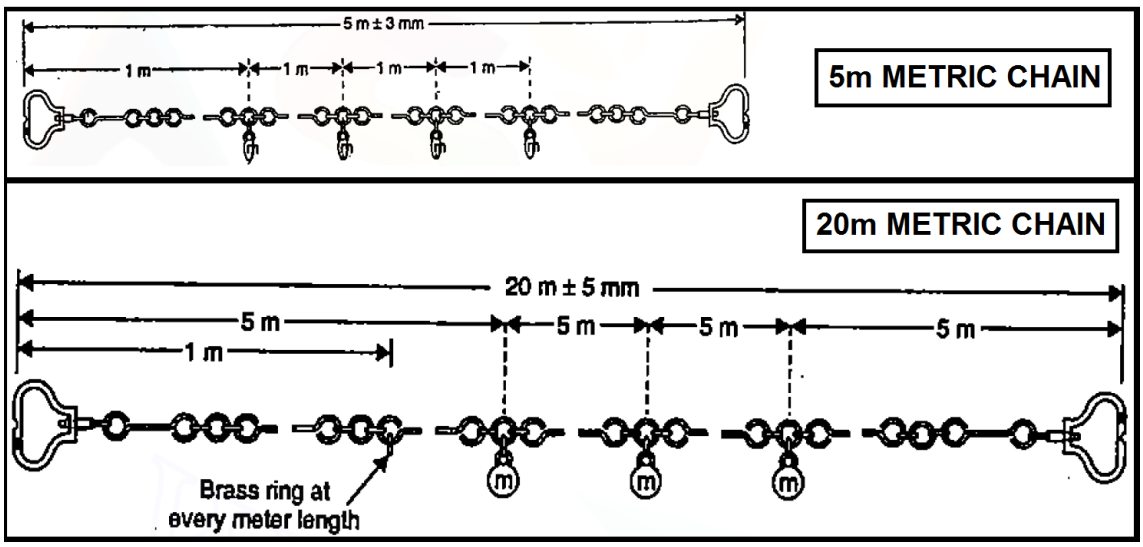
$$\mathbf{D} = \frac{\mathbf{S}}{2\ \tan(\frac{\theta}{2})}$$

- Besides only being suitable for relatively short lines, this method of distance measurement is time consuming and is not used today.

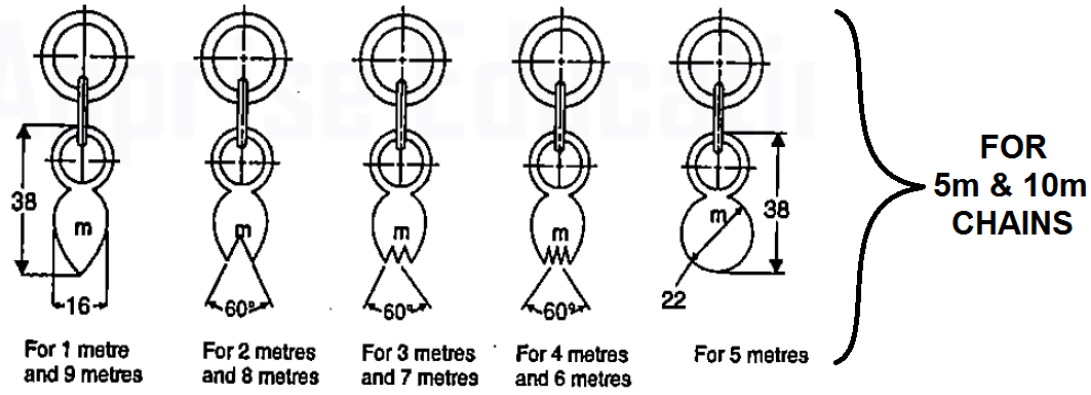
6) Chain

- **Chain** is the primary instrument used for linear measurements in surveying.
  - It is composed of 100 or 150 pieces of mild steel wire 4mm diameter called links.
  - Ends of each link is bend to form loops. They are connected to each other by 3 oval rings.
  - Ends of chain are provided with brass handles, each with a swivel joint. The length of the chain is measured from the outside of the handle.
  - Different types of chain used are:
    - Metric Chain
    - Non-metric Chain – Gunter's chain, Revenue Chain, Engineers' Chain

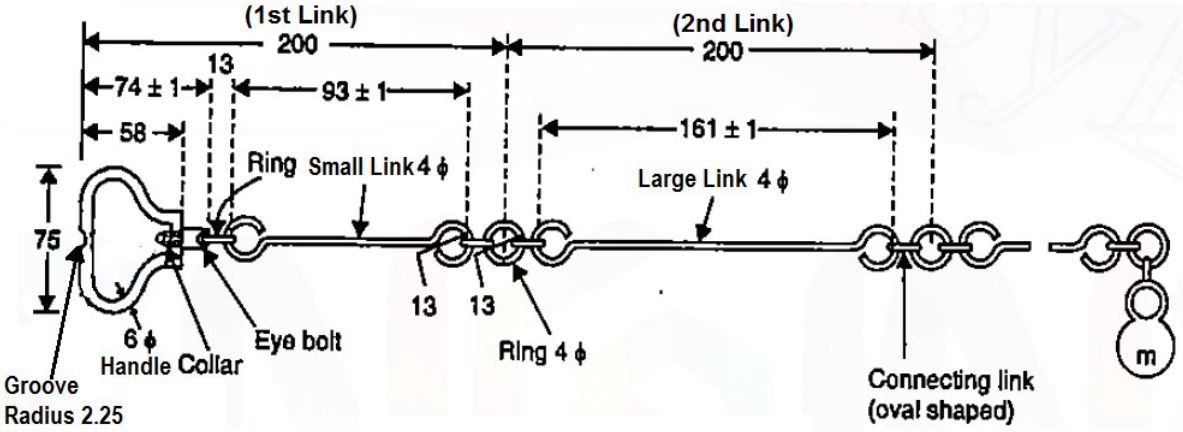
- **Metric Chain:**
  - Used in India, based on IS 1492 – 1970
  - Available in lengths of 5m, 10m, 20m & 30m



  - To ease the reading of chain, tallies are fixed at:
    - Every metre length – for 5m & 10m chains
    - Every 5m length – for 20m & 30m chains



  - In 20m & 30m chains, brass rings are provided at every metre lengths, except where tallies are provided.
  - Details of a metric chain is shown here *(All dimensions in mm)*



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- **Non-metric Chains**
  - ***a) Gunter's chain***
    - Length : 66ft
    - Links : 100nos
    - Used in measuring distances in miles, furlongs etc. and areas in acres. This is because,
      - 10 Gunters' chains = 1 furlong
      - 80 Gunters' chains = 1 mile
      - 10 square chains = 1 acre
  - ***b) Revenue Chain***
    - Length : 33ft
    - Links : 16nos
    - Used in cadastral surveys
  - ***c) Engineers' Chain***
    - Length : 100ft
    - Links : 100nos
    - Used in measuring distances in feet. Use generally for engineering surveys.

- **Testing Of Chain**
  - The accuracy of a chain subjected to a tension of 8kg and checked against a certified steel tape (standardized at 20^{o}C) shall satisfy the following:
  - Overall length should be within:
    - For 20m chain: ± 5mm
    - For 30m chain: ± 8mm
  - Every metre length of chain should be accurate to within 2mm.

7) Tape

(a) Linen tape or Cloth Tape

- It is a varnished strip made of closely woven linen.
- The width of the strip is about 12 to 16 mm. It is available in different lengths such as 10m, 20m, 30m, and 50m.
- Both ends of the tape are provided with metallic handles and the whole tape is wounded in leather or metal case.
- Light in weight and easy to handle. It may shrink when exposed to water and also elongate when pulled.
- Not suitable for accurate surveying measurements. These are generally used for measuring offsets and for ordinary works.

(b) Woven Metallic Tape

- The metallic woven tape is an improved version of linen tape. Brass or copper made wires are used as reinforcement for the linen material. Hence, it is more durable than normal linen tape.
- Metallic handles are provided at the end of the tape which is included in the length of the tape.
- They are available in different lengths of 2m, 10m, 15m, 20m, 30m, and 50m.
- These are used for survey works such as topographical survey works where minor errors are not taken into consideration.

(c) Steel Tape

- Made of steel or stainless steel.
- It consists of a steel strip of 6mm to 16mm wide.
- Available in lengths of 1m, 5m, 8m, 10m, 20m, 30m and 50m.
- It generally comes a metal case with automatic winding device.
- The tape is withdrawn from the case by using a hand during measuring and it is rewound into the case by just pressing button provided on the case.
- Steel tapes are not flexible and are suitable for measuring leveled surfaces only.
- They may corrode easily when exposed to moisture and to prevent this tape, it should be cleaned and oiled after every use.
- These tapes are generally used for standardizing chains, measurements of constriction works, etc.

(d) Invar Tape

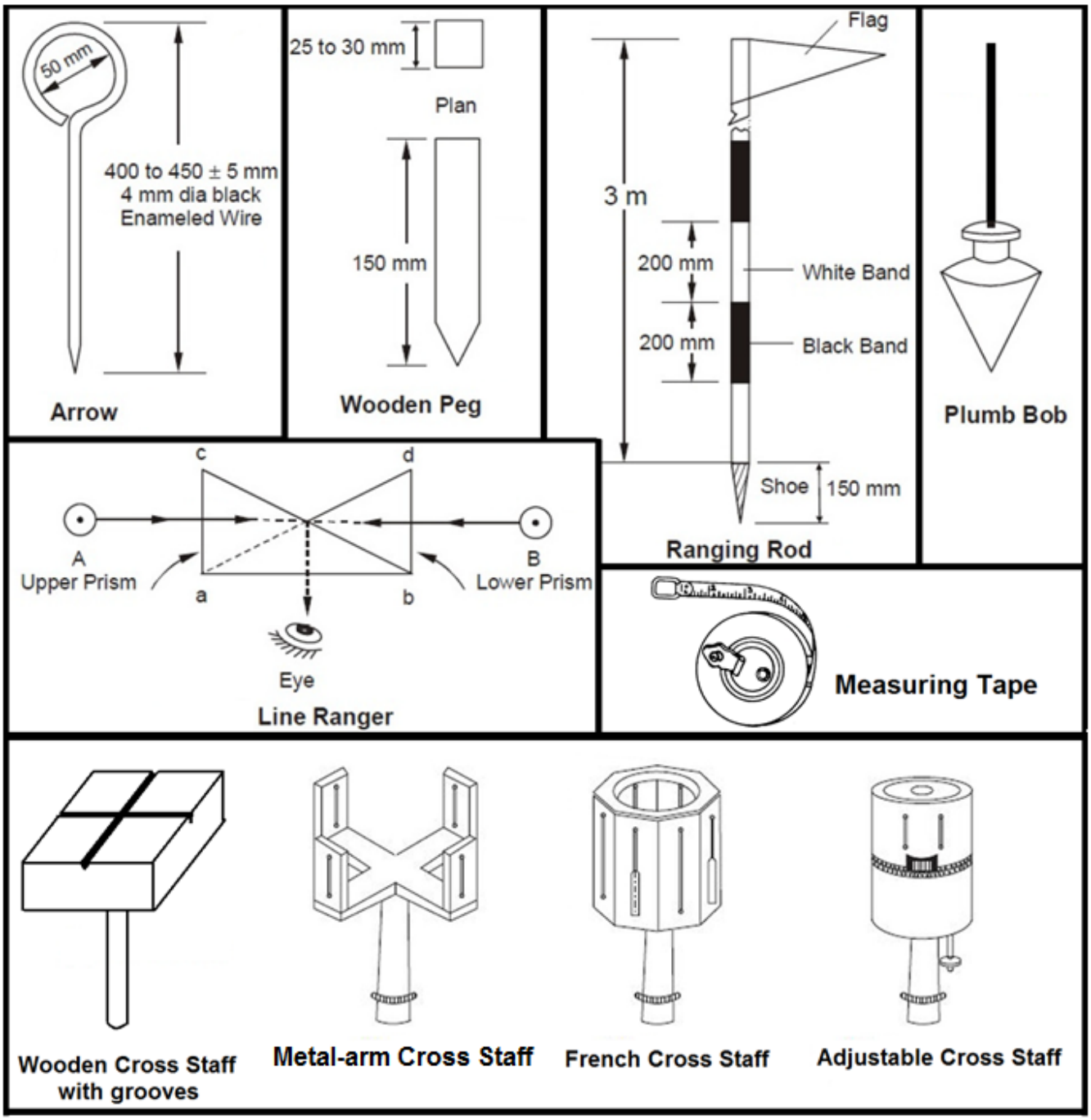
- Made of a special nickel–steel alloy (35% nickel and 65% steel) to reduce length variations caused by differences in temperature.
- The thermal coefficient of expansion and contraction of this material is only about 1/30 to 1/60 that of an ordinary steel tape.
- However, the material is soft and somewhat unstable. This weakness, along with the cost perhaps ten times that of steel tapes, made them suitable only for precise geodetic work and as a standard for comparison with working tapes.

(e) Fiberglass Tape

- It is a synthetic tape, made of fiber-reinforced plastic using glass fiber.
- These are light in weight and flexible.
- Available in lengths of 5m, 10m, 20m, 30m, and 50m.
- It may stretch when subjected to tension. Hence, these are not suitable for accurate surveying works.
- However, synthetic tapes are recommended in place of steel tapes where it is essential to take measurements in the vicinity of electric fences and railway lines, etc.

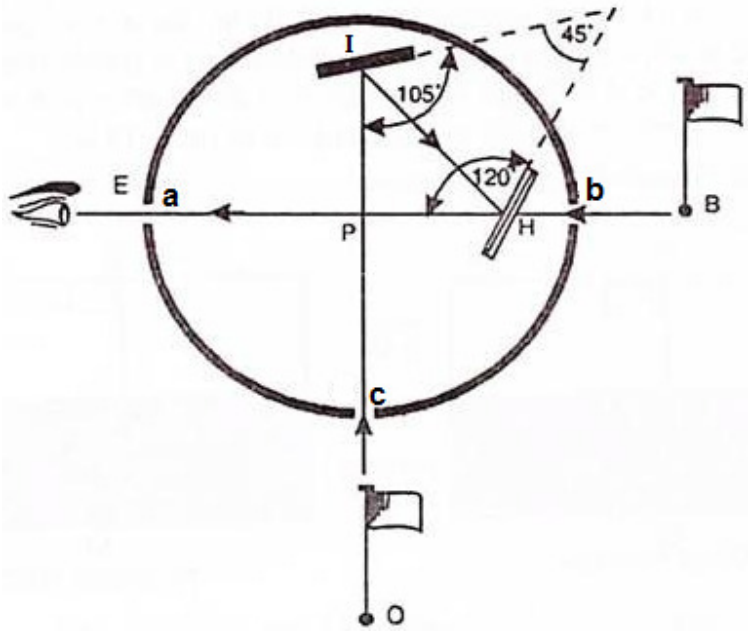
CHAINING & TAPING ACCESSORIES

1) ***Arrows*** or ***chain pins*** are used to mark the position of the ends of the chain on the ground.
2) ***Wooden pegs*** are used to mark the positions of the survey stations or the end points of a survey line. The typical dimensions are 25 mm x 25 mm in cross section and 150 mm long with a nail at the top.
3) ***Ranging rods or poles*** are used to make measurements along a straight line.
4) ***Plumb bobs*** are used to project a point on the ground upto the tape or to project a point on the tape down to the ground.
5) ***Line ranger*** is an optical instrument used for locating a point on a line and hence useful for ranging (i.e., establishing intermediate station points on a survey line). It consists of two isosceles prisms placed one over the other and fixed in an instrument with handle. The diagonals of the prisms are silvered so as to reflect the rays. Its advantage is that it needs only one person to range.
6) ***Cross staff*** is used for setting out right angles and finding the foot of perpendicular from the object on the lines.
   - The simplest form of cross-staff is the ***open wooden cross-staff***. It consists of a round or square piece of wood about 4 cm thick and varying form 15 cm to 30 cm in diameter or side mounted on an iron shod wooden staff about 2.5 cm diameter and 1.5 m long. The disc is provided with two saw cuts about 1 cm deep at right angles to each other, giving two lines of sight.
   - The modified form of the open cross-staff is the ***metal-arm cross-staff*** in which the wooden head is replaced by four metal arms with vertical slits for sighting through at right angles to each other
   - The ***French cross-staff*** consists of an octagonal brass tube with slits on all the eight sides. It has an alternate vertical sighting slit and an opposite vertical window with a vertical fine wire or horse hair on each of the four sides. These are used for setting out right angles. On the other sides are vertical slits, which are at 45° to those mentioned above for setting out angles of 45°. The sights being too close, it is not as accurate as the open cross-staff.
   - The ***Adjustable cross staff*** consists of two cylinders of equal diameter placed one over each another and is used to set out any angle.



7) ***Optical Square*** is more accurate than the cross-staff and is used for setting out accurately the long offsets. It is a small compact hand instrument based upon the principle of reflection.
   - It is a round brass box about 5 cm in diameter and 1.25 cm deep.
   - H and I are two mirrors placed at an angle of 45° to each other. The mirror H, known as the horizon mirror, is half silvered and half unsilvered and is rigidly attached to the base plate of the box. The mirror I, called the index mirror, is wholly silvered and is attached to the box in such a way that it can be adjusted to any position.

- Three openings are made in the rims of the box: a is a small hole for the eye ; b is a small rectangular slot for the horizon sight placed diametrically opposite to the eye-hole ; and c is a large rectangular slot for index sight placed at right angles to the line joining a and b.



RANGING OUT SURVEY LINES

- The process of fixing or establishing intermediate points to facilitate measurement of the survey lines are called as Ranging. The intermediate points are located by means of ranging rodes, offset rods and ranging poles.
- While measuring the survey lines, the chain or the tape has to be stretched along the survey line along that joins two terminal stations. When the line to be measured has a smaller length compared to the chain, then the measurement goes smooth. If the length of the line is greater, the survey lines have to be divided by certain intermediate points, before conducting the chaining process. This process is called ranging.
- The process of ranging can be done by two methods:
    1. Direct Ranging
    2. Indirect Ranging

1. Direct Ranging

Direct ranging is the ranging conducted when the intermediate points are intervisible. Direct ranging can be performed by eye or with the help of an eye instrument (Line Ranger).

- **Ranging by Eye**
    - Let A and B are the two intervisible points at the ends of the survey line. The surveyor stands with a ranging rod at the point A by keeping the ranging rod at the point B. The ranging rod is held at about half metre length.

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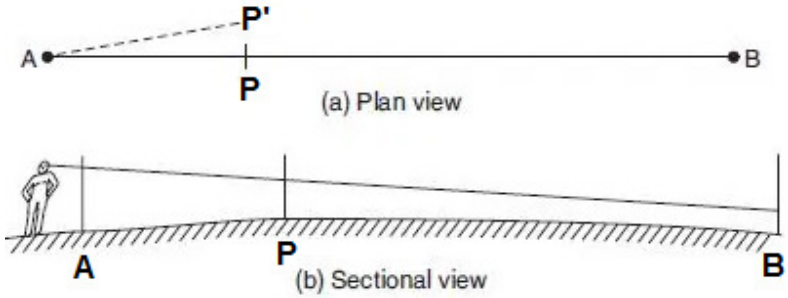
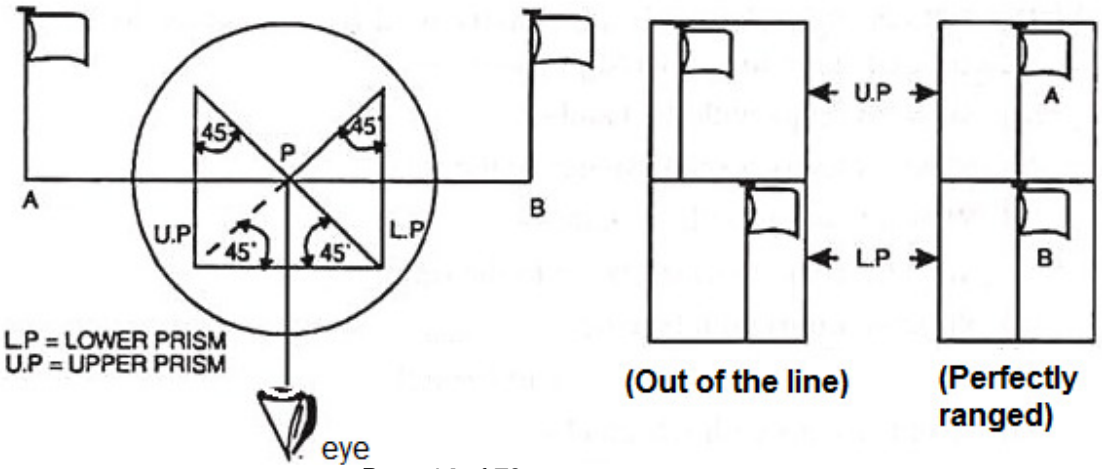


- The assistant then takes the ranging rod and establishes at a point in between AB, almost in line with AB. This is fixed at a distance not greater than one chain length from point A.
- The surveyor can give signals to the assistant to move traverse till the rod is in line with A and B. In this way, other intermediate points are determined.
- Code of signals used here:

Sl. No.	Signal by the Surveyor	Action by the Assistant
1	Rapid sweep with right hand	Move considerably to the right
2	Slow sweep with right hand	Move slowly to the right
3	Right arm extended	Continue to move to the right
4	Right arm up and moved to the right	Plumb the rod to the right
5	Rapid sweep with left hand	Move considerably to the left
6	Slow sweep with left hand	Move slowly to the left
7	Left arm extended	Continue to move to the left
8	Left arm up and moved to the left	Plumb the rod to the left
9	Both hands above head and then brought down	Correct
10	Both arms extended forward horizontally and the hands depressed briskly	Fix the rod

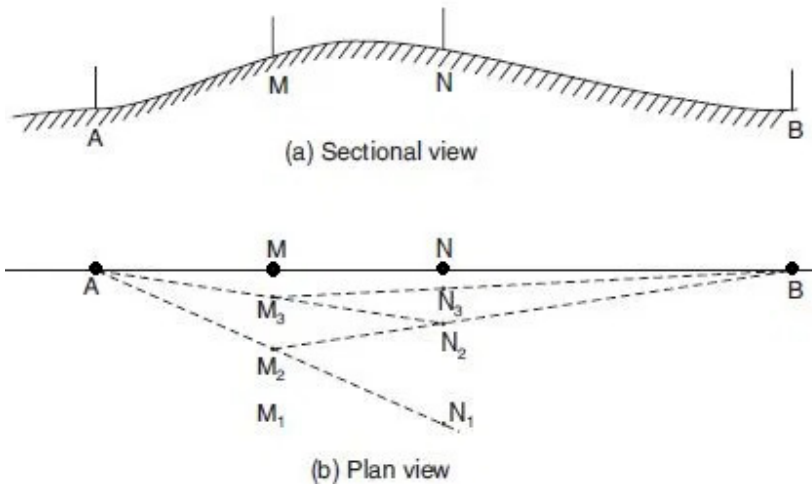
Ranging by Line Ranger

- Line Ranger is used to range out survey lines between intervisible points A and B. The surveyor moves the instrument till the two images come in the same vertical line. With the help of a plumb bob, the point P is then transferred to the ground. This instrument can be used to locate the intermediate points without going to the other end of the survey line. This requires only one person to range.



2. Indirect Ranging or Reciprocal Ranging

- Indirect ranging is employed when the two points are not intervisible or the two points are at a long distance. This may be due to some kind of intervention between the two points. In this case, the following procedure is followed.
- As shown in figure-3, two intermediate points are located M1 and N1 very near to chain line by judgment such that from M1, both N1 and B are visible & from N1 both M1 and A are visible.



- At M1 and N1 two surveyors stay with ranging rods. The person standing at M1 directs the person at N1 to move to a new position N2 as shown in the figure. N2 must be inline with M1B.
- Next, a person at N2 directs the person at M1 to move to a position M2 such that it is inline with N2A. Hence, the two persons are in points are M2 and N2.
- The process is repeated until the points M and N are in the survey line AB. Finally, it reaches a situation where the person standing at M finds the person standing at N in line with NA and vice versa. Once M and N are fixed, other points are fixed by direct ranging.

SURVEYING USING CHAIN

Characteristics of Chain surveying:

- Chain survey is suitable in the following cases:

  (i) Area to be surveyed is comparatively small

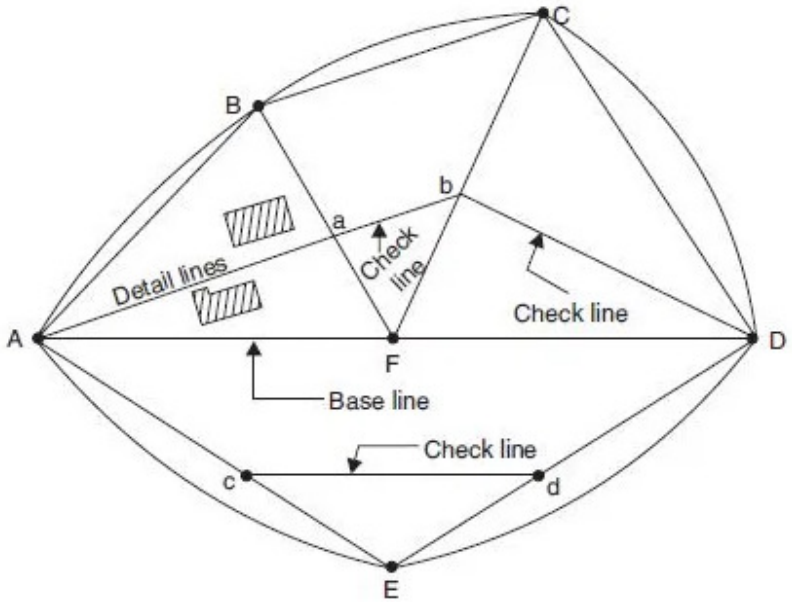
  (ii) Ground is fairly level

  (iii) Area is open and

  (iv) Details of plot are simple and less.

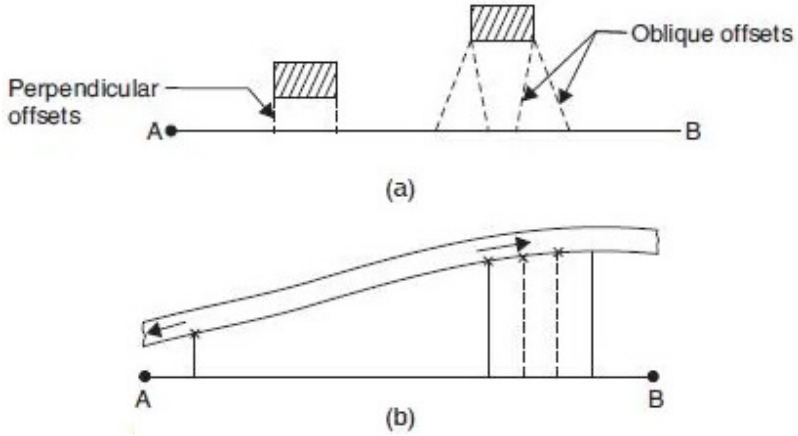
- In chain surveying, only linear measurements are made i.e. no angular measurements are made.
- Since triangle is the only figure that can be plotted with measurement of sides only, in chain surveying the area to be surveyed should be covered with a network of triangles.

- Figure below shows a typical scheme of covering an area with a network of triangles. No angle of the network triangles should be less than 30º to precisely get plotted position of a station with respect to already plotted positions of other station. As far as possible angles should be close to 60º. However, the arrangement of triangles to be adopted depends on the shape, topography, natural and artificial obstacles in the field.



Terminologies used in Chain Surveying:

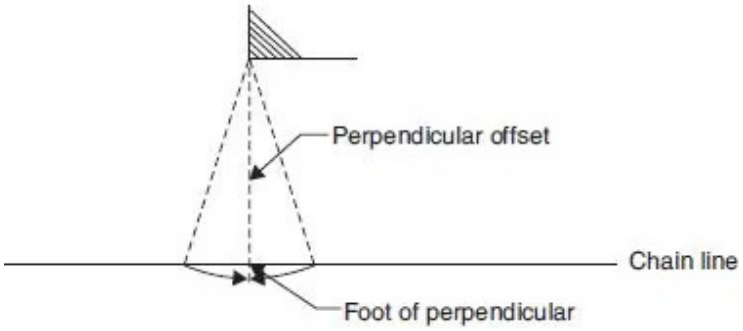
a) ***Station:*** Station is a point of importance at the beginning or at the end of a survey line.
b) ***Main station:*** These are the stations at the beginning or at the end of lines forming main skeleton. They are denoted as A, B, C etc.
c) ***Subsidiary or tie stations:*** These are the stations selected on main lines to run auxiliary/secondary lines for the purpose of locating interior details. These stations are denoted as a, b, c, …., etc., or as 1, 2, 3, … etc.
d) ***Base line:*** It is the most important line and is the longest. Usually it is the line plotted first and then framework of triangles are built on it.
e) ***Detail lines:*** If the important objects are far away from the main lines, the offsets are too long, resulting into inaccuracies and taking more time for the measurements. In such cases the secondary lines are run by selecting secondary stations on main lines. Such lines are called detail lines.
f) ***Check lines:*** These are the lines connecting main station and a substation on opposite side or the lines connecting to substations on the sides of main lines. The purpose of measuring such lines is to check the accuracy with which main stations are located.
g) ***Offsets:*** Lateral measurements to chain lines for locating ground features are known as offsets.



- For this purpose, *perpendicular* or *oblique offsets* may be taken (see figure (a) above). If the object to be located (say road) is curved more number of offsets should be taken (see figure (b)). For measuring offsets, tapes are commonly used. For setting perpendicular offsets any one of the following methods are used:

  (i) Swinging

  Chain is stretched along the survey line. An assistant holds the end of tape on the object. Surveyor swings the tape on chain line and selects the point on chain where offset distance is the least and notes chain reading as well as offset reading in a field book on a neat sketch of the object.



  (ii) Using cross staff

  Cross-staff is held vertical on the chain line at a point where the foot of the offset is likely to occur. Through one pair of slits, the ranging rod at the end of the survey line is sighted. Looking through the other pair of slits, it is seen whether the point to which offset is to be taken is bisected. If not, the cross-staff is moved forward or backward till the point is bisected.

  (iii) Using optical square

  Procedure is similar to that of cross-staff.

Selection of Stations:

The following points should be considered while selecting station points:

(i) It should be visible from at least two or more stations.

(ii) As far as possible main lines should run on level ground.

(iii) All triangles should be well conditioned (No angle less than 30º).

(iv) Main network should have as few lines as possible.

(v) Each main triangle should have at least one check line.

(vi) Obstacles to ranging and chaining should be avoided.

(vii) Sides of the larger triangles should pass as close to boundary lines as possible.

(viii) Tresspassing and frequent crossing of the roads should be avoided.

Procedures in Chain Surveying

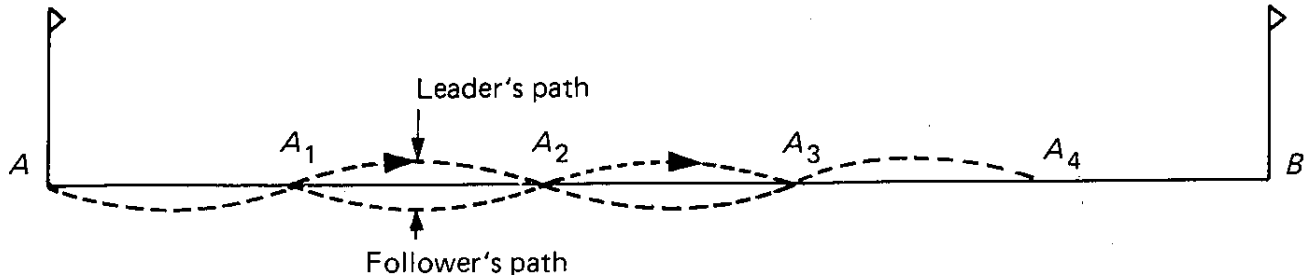
1. Unfolding Of Chain:

To open a chain, the strap is unfastened and the two brass handles are held in the left hand and the bunch is thrown forward with the right hand. Then one chainman stands at the starting station by holding one handle. The other chainman moves forward by holding the other handle forward until the chain is completely extended.

2. Measuring the length of a line:

- The operation is carried out by two assistants known as Chainmen. One acts as Leader, and the other as Follower.
- The chainman at the forward end of the chain, who drag the chain forward, is known as Leader. The duties of the leader are as follows:
    a) To drag the chain forward with some arrows and a ranging rod.
    b) To fix arrows on the ground at the end of every chain.
    c) To obey the instructions of the follower.
- The chainman at the rear end of the chain, who holds the zero end of the chain at the station, is known as the Follower. The duties of the follower are :
    a) To direct the leader at the time of ranging.
    b) To carry the rear handle of the chain.
    c) To pick up the arrows inserted by the leader.
- Before starting the chaining operation, two ranging rods should be fixed on the chain line, at the end stations A and B (as shown in figure). The other ranging rods, should be fixed near the end of each chain length (A_1, A_2,….), during the ranging operation.
- To chain the line, the leader moves forward by dragging the chain and by taking with him a ranging rod and 10 arrows . The follower stands at the starting station by holding the other end of chain.
- When the chain is fully extended , the leader holds the ranging rod vertically at arm's length, over the point at the end of the chain. The follower directs the leader to move his rod to the left or right until the ranging rod is exactly in line (*Ranging by eye*). Then the

follower holds the zero end of the chain by touching the station peg. The leader stretches the chain and then inserts an arrow on the ground at the end of the chain.

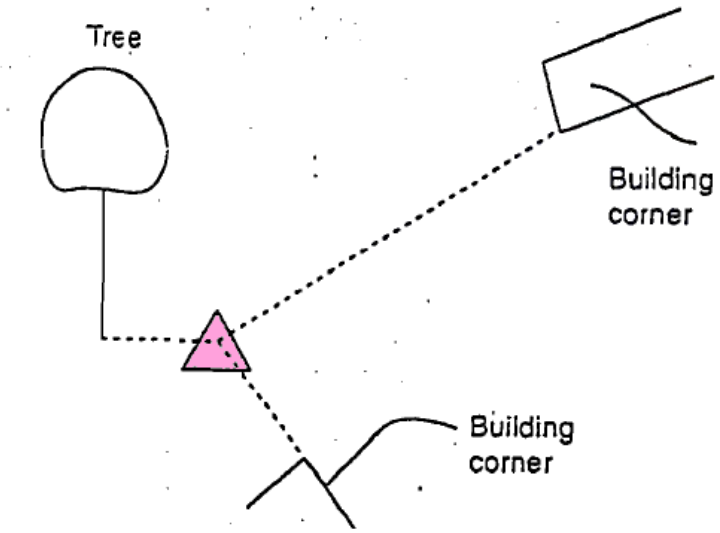


- Again, the leader moves forward by dragging the chain with nine arrows and the ranging rod. At the end of the chain, he fixes another arrow as before. As the leader moves further, the follower picks up the arrows which were inserted by the leader.
- In this way, chaining is continued. When all the arrows have been inserted and the leader has none left with him, the follower hands them over to the leader; this should be noted by the surveyor.
- If the remaining fractional length is to be measured, the leader should drag the chain beyond the station and the follower should hold the zero end of the chain at the last arrow. Then the odd links should be counted.

3. Recording field notes:

The data obtained in the field are recorded systernatically in an oblong book of size about 200 mm by 120rnrn which is known as field book. It opens lengthwise and usually has two lines spaced about 15 to 20 mm apart ruled down the middle of each page. This is known as *double line field book* and the distance along the chain line is entered within the double lines. The important steps before starting a survey are:

i) Make a rough sketch in the field book showing the locations of chosen stations and chain lines.
ii) The bearing from true or magnetic north of atleast one of the lines should be shown.
iii) The stations should be located from three or more points and enough information should be plotted so that they can be relocated if necessary.

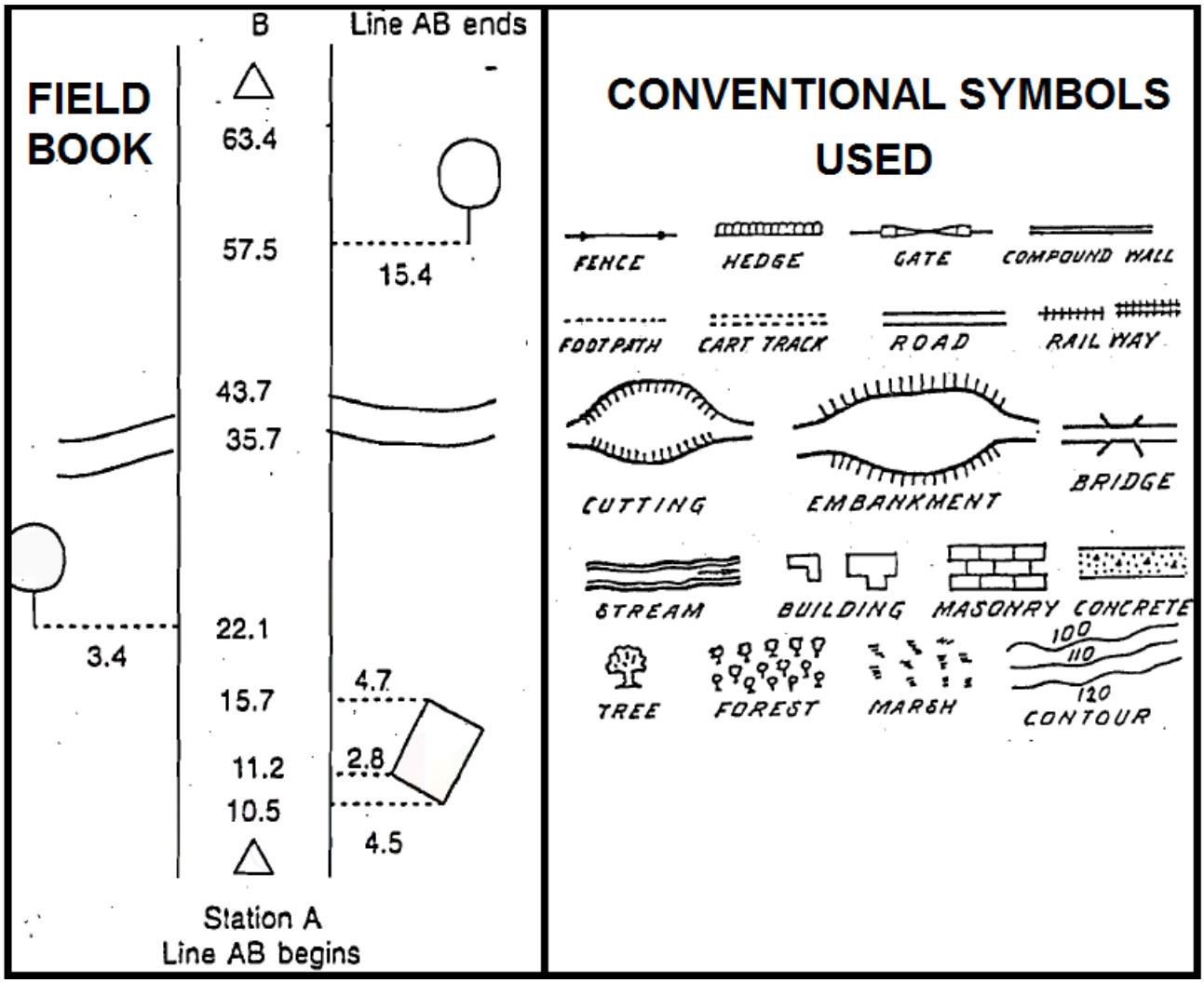


Fixing of a station from atleast 3 permanent structures

Following are the guide lines in recording a field book:

- Begin each line at the bottom of a page.
- Sufficient space should be kept in the field book between different chainages. Plotting in the field book need not be to scale.
- Small details should be plotted in an exaggerated scale.
- Clear sketches of all details should be shown in the field book. Nothing should be left to memory..
- Bookings should be done systematically starting with the side having more details.

A typical page of a double line field book and the set of conventional symbols used are shown below.



4. Folding of Chain :

After the completion of the work, the chain should be folded in to a bundle and fastened with a leather strap. To do this, the handles of the chain should be brought together by pulling the chain at the middle. Commencing from the middle, take two pairs of link at a time with the right hand and place them obliquely across the other in the left hand. When the chain is collected in a bundle, it is tied with a leather strap. This process is called the folding of chain.

Chaining on Sloping ground

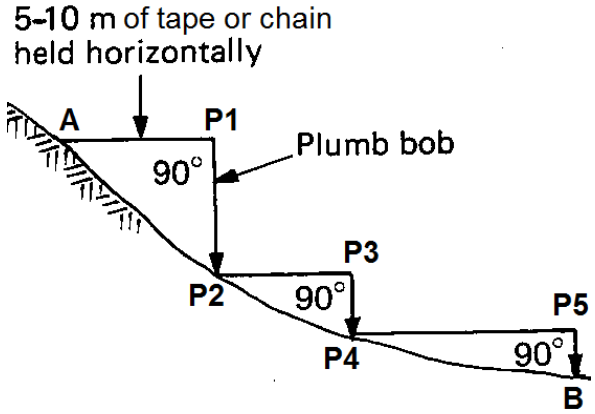
Chaining along the surface of a sloping ground gives only the sloping distance. For plotting the surveys, horizontal distances are required. It is therefore, necessary to determine the horizontal distances between the stations. The following are the different methods that are generally employed.

1) Direct Method or Stepping Method

2) Indirect Method

1) Direct Method or Stepping Method

- The line AB is first ranged properly.
- Then, the follower holds the zero end of the tape at A.
- The leader selects a suitable length AP1 so that P_1 is at chest height and AP_1 is just horizontal. (The horizontality is maintained by eye estimation, by tri-square or by wooden set-square.)
- The point P_2 is marked on the ground by plumb-bob so that P_1 is just over P_2.
- The horizontal length AP_1 is noted then the follower moves to the position P_2 and holds the zero end of the tape at that point. Again the leader selects a suitable length P_2P_3 in such a way that P_2P_3 is horizontal and P_4 is marked on the ground by dropping plumb bob. Similarly P_4P_5 is also measured.
- Hence, the total horizontal length, $AB = AP_1 + P_2P_3 + P_4P_5$

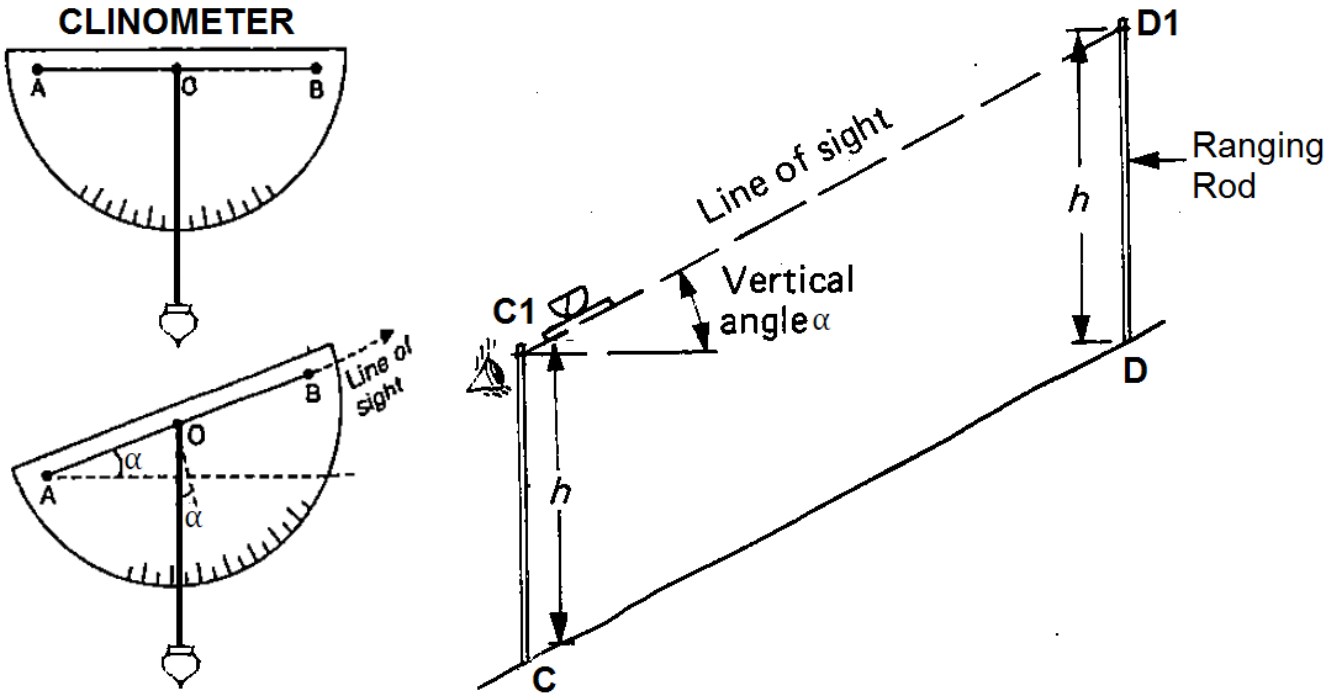


2) Indirect Method

When the slope of the ground surface is long and gentle, the stepping method is not suitable. In such a case, the horizontal distance may be obtained by the indirect methods, such as:

a) Measuring slope with a clinometer :

- Clinometer is a graduated semicircular protractor. It consists of two pins A and B for sighting the object. A plumb bob is suspended from point O with a thread. When the line of sight AB is just horizontal, the thread passes through 0^o. When the line of sight is inclined, the plumb line remains vertical, but passes through a graduation on the arc which shows the angle of inclination.



- Suppose C and D are two points on sloping ground. Two ranging rods are fixed at these points. Then two other points C1 and D1 are marked on the ranging rods so that CC1 = DD1.
- Now, clinometer is placed in such a way that its centre just touches the mark C1. The clinometers is then inclined gradually until the points A, B, and D1 are in the same straight line. At this position, the plumb line of the clinometers will show an angle which is the angle of slope α of the ground. The sloping distance CD is also measured.

The required horizontal distance = CB = CD cosα

b) Measuring slope applying Hypotenusal Allowance :

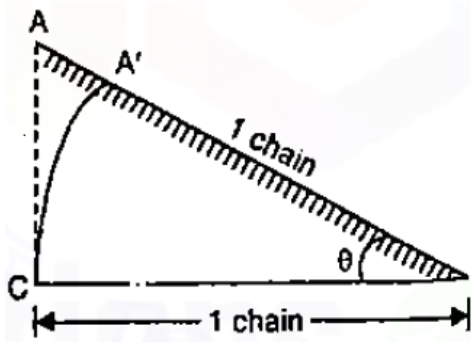
- In this method , the slope of the ground is first out by using the clinometers. When the chain is stretched along the slope (AB), arrow is not driven at the end of the chain (A'), but advanced to point A on the ground, such that the horizontal equivalent of sloping distance AB represents one chain length. This additional length AA' is known as *Hypotenusal allowance*.

Let Θ = angle of slope measured by clinometers
A'B = CB = one chain length = 100 links

$\sec \Theta = \frac{AB}{CB} = \frac{AB}{100}$

=> AB = 100 sec Θ

AA' = AB – A'B
= 100 sec Θ – 100
= <u>100 (sec Θ – 1)</u>



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ERRORS IN LINEAR MEASUREMENTS / CHAINING

ERRORS in chaining may be caused due to variation in temperature and pull, defects in instruments etc. They may be classified into two categories.

(i) Compensating errors

(ii) Cumulative errors

(i) COMPENSATING ERRORS: Errors, which may occur in both directions (that is both positive and negative) and which finally tend to compensate are known as compensating errors.

Sources: (a) Careless holding of chain

(b) Careless marking of points

(c) Pull applied on the chain is not equal to standard pull, but pull is not measured (sometimes more, sometimes less)

(ii) CUMULATIVE ERRORS: Errors, which may occur in the same direction and which finally tend to accumulate are said to be cumulative. They seriously affect the accuracy of the work and are proportional to the length of the line (L).The errors may be positive or negative.

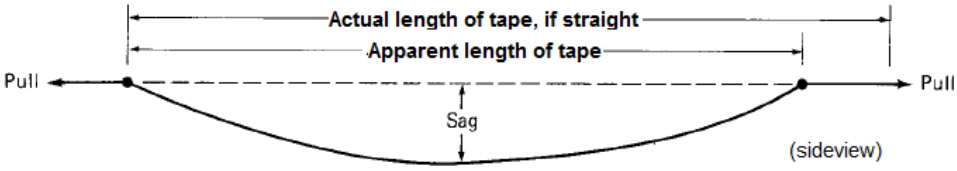
- **Positive Cumulative Error:** The error, which make the measured length more than the actual is known as positive cumulative error.

Sources: (a) The length of chain is shorter than its standard length due to:

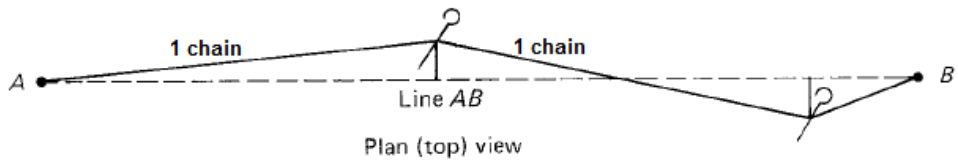
- Bending of links
- Knots in connecting links.
- The field temperature is lower than that at which the tape was calibrated.
- The pull on the chain at field is lower than the standard pull
- Shrinkage of tape when moist
- Removal of too many rings due to adjustment of its length.

(b) Slope correction is not considered while measuring along slope.

(c) Sag correction is not applied when chain is suspended from ends.



(d) Bad Ranging



(e) Bad straightening of chain

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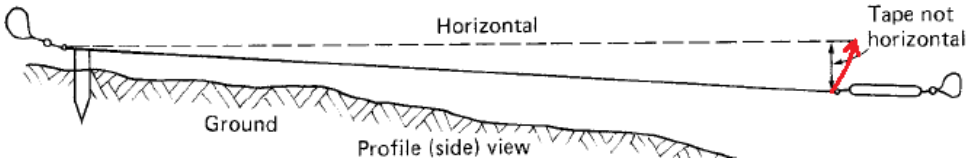
(f) Non-horizontality of chain



- **Negative Cumulative Error:** The error, which make the measured length less than the actual is known as negative cumulative error.

  ***Sources*:** The length of chain is longer than its standard length due to:

  - Flattening of connecting rings.
  - Opening of the ring joints.
  - Field temperature is higher than that at which the tape was calibrated.
  - The pull on the chain at field is higher than the standard pull

MISTAKES: Errors occurring due to the carelessness of the chainman are called *mistakes* or *human errors*. Following are a few common mistakes:

(1) Displacement of arrows: Once an arrow is withdrawn unintentionally from the ground during chaining and not replaced in proper position.

(2) Miscounting of chain length: A full chain length may be omitted or added. This happen when arrows are lost or wrongly counted.

(3) Misreading: The number may be read from the wrong direction etc.

(4) Erroneous booking: Some number may be called or entered wrongly.

PRECAUTIONS AGAINST ERRORS AND MISTAKES:

(1) The point where the arrow is fixed on the ground should be marked with a cross(×).

(2) The zero end of the chain or tape should be properly held.

(3) During chaining the number of arrows carried by the follower and leader should always tally with the total number of arrows taken.

(4) The chainman should call the measurement loudly and distinctly and the surveyor should repeat them while booking.

(5) Ranging should be done accurately.

(6) No measurement should be taken with the chain in suspension.

CORRECTIONS IN LINEAR MEASUREMENTS

To eliminate the errors discussed above, following corrections are applied to chain or tape measurements.

(i) Correction for incorrect length or *Standard length correction*
(ii) Temperature correction
(iii) Pull correction
(iv) Sag correction
(v) Slope correction

(i) Correction for incorrect length or Standard length correction:

An error, caused by incorrect length of a tape, occurs each time the tape is used.

$$C_L = \left(\frac{l - l'}{l'}\right)L$$

where C_L is the correction to be applied to the measured (recorded) length of a line to obtain the true length, l the actual tape length on field, l' the designated tape length, and L the measured (recorded) length of line.

(ii) Temperature correction:

Tapes are standardized at definite temperatures. A temperature higher or lower than this value causes a change in length that must be considered.

$$C_T = k(T_1 - T)L$$

where C_T is the correction in the length of a line caused by nonstandard field temperature, k the coefficient of thermal expansion and contraction of the tape, T_1 the tape temperature at the time of measurement (non-standard temperature), T the tape temperature when it has standard length (standard temperature), and L the measured (recorded) length of line.

(iii) Pull correction:

When a tape is pulled with a tension greater than its standard pull (the tension at which it was calibrated), the tape will stretch and become longer than its standard length. Conversely, if less than standard pull is used, the tape will be shorter than its standard length.

$$C_P = (P_1 - P)\frac{L}{AE}$$

where C_P is the total elongation in tape length due to pull; P_1 the pull applied to the tape at the time of the observation; P the standard pull for the tape; A the cross-sectional area of the tape; E the modulus of elasticity of steel; and L the measured (recorded) length of line.

(iv) Sag correction:

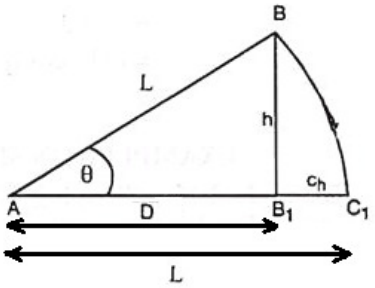
A steel tape not supported along its entire length sags in the form of a *catenary*, a good example being the cable between two power poles. Because of sag, the horizontal distance (chord length) is less than the graduated distance between tape ends.

$$C_S = -\frac{w^2 L_S^3}{24 P_1^2}$$

where C_S is the correction for sag (difference between length of sagged tape and the actual length of tape if held straight), in feet; L_S the unsupported length of the tape; *w* the weight of the tape per unit length (in kg/metre); and *P*1 the pull on the tape (in kg).

(v) Slope correction:

The distance measured along the slope between two stations is always greater than the horizontal distance between them. The difference in slope distance and horizontal distance is known as slope correction which is always substractive.



Let L = slope distance AB

D = horizontal distance AC

h=difference in reduced levels of A and B

$$D = \sqrt{(L^2 - h^2)}$$

$$\text{Slope Correction} = L - D = \frac{h^2}{2L}$$

CHAPTER 3

PLANE TABLE SURVEYING

PURPOSE OF PLANE TABLE SURVEYING

- It is a graphical method of survey in which the field observations and plotting of plan shall proceed simultaneously.
- It is a quick method, where the necessity of transferring the field data to the office for map-making is completely avoided. Here plotting of plan or map is performed in-situ.
- Further, the process of recording and transcribing field notes is completely eliminated

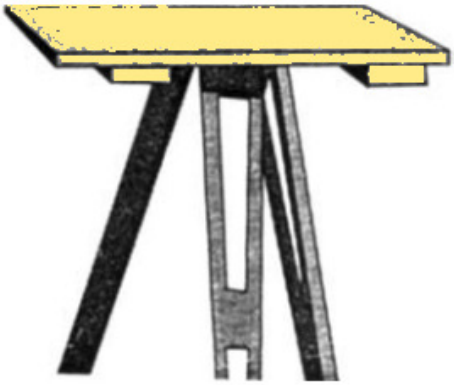
INSTRUMENTS USED

The following instruments are used in plane table survey:

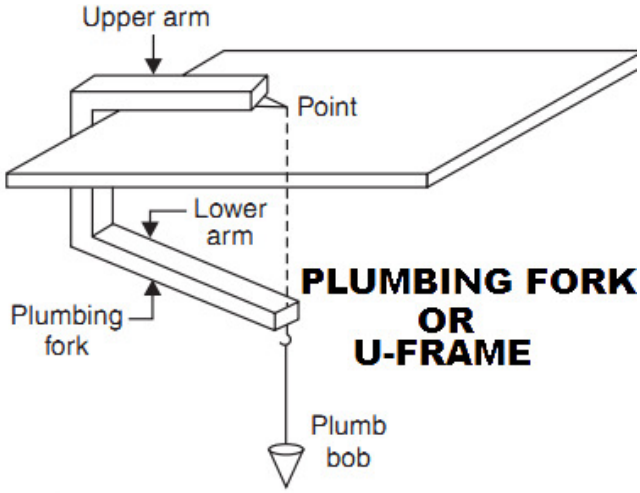
1. The plane table with levelling head having arrangements for *(a)* levelling, (b) rotation about vertical axis, and (c) clamping in any required position.
2. Alidade for sighting
3. Plumbing fork and plumb bob.
4. Spirit level.
5. Compass.
6. Drawing sheet

1. **Plane table**
   - It consists of a small drawing board mounted on light tripod in such a way that the boards can be rotated about the vertical axis and can be clamped in any position.

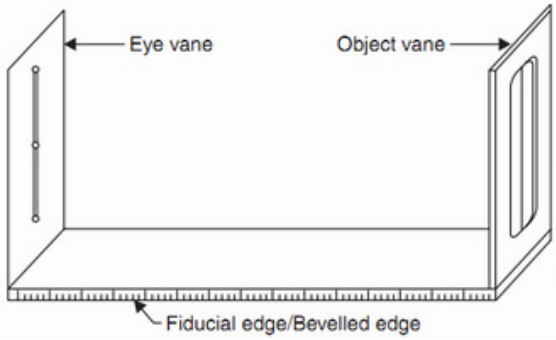
2. **Alidade (or sight rule)**
   - This may be (i) PIain Alidade (generally used - shown in figure) or (ii) Telescopic alidade.
   - The front or object vane and the rear or sight vane are of folding type. It is possible to suitably clamp them in vertical position. The front vane has a thread across its length andthe rear vane has a fine slit to view through it.
   - Alidade can be rotated about the point representing the instrument station on the drawing sheet over the plane table, so that the line of sight passing through the object can be represented on the sheet by drawing a line against the working edge of the alidade (known as *fiducial edge*), which is beveled in nature.



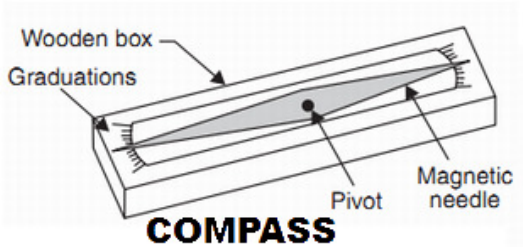
PLANE TABLE



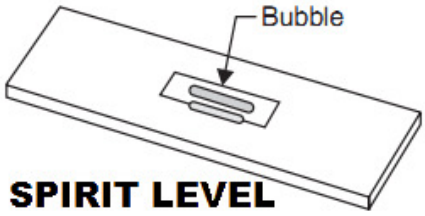
PLUMBING FORK OR U-FRAME



ALIDADE



COMPASS



SPIRIT LEVEL

3. **Plumbing fork or U-frame**
   - This is a U-shaped metal frame with two parallel arms of equal length, a plumb bob being attached to the free end of the lower arm.
   - The point of the upper arm and the plumb line are in the same vertical line.
   - The plumbing fork is used for transferring the point on the ground to the sheet or from the sheet to the ground (c*entring the table*).

4. **Spirit level**
   - A spirit level (either of the tubular variety or of the circular type) is used to check whether the plane table is level.
   - This can be ascertained by placing the level in two directions at right angles to each other and observing that the bubble is central in both cases.

5. **Compass**
   - It is used to orient the plane table to magnetic North.
   - Usually a trough compass is used.

WORKING OPERATIONS WITH PLANE TABLE

Before mapping work can start with plane table, the following preliminary steps are performed:

1. Fixing: The plane table should be fixed to the tripod. The working sheet should be carefully mounted with thumb screws.

2. Levelling: For small works, the plane table is leveled through eye estimation. For more accurate works, spirit level is to be used. Here, the spirit level is placed parallel to any two legs of the tripod and the bubble is made central by adjusting these legs. Now the spirit level is placed in the direction perpendicular to it and the bubble is brought central by adjusting the third leg of tripod.

3. Centring: The table should be so placed that the plotted point '*a'* is exactly over the ground point 'A'. This is done using plumbing fork.

4. Orientation: It is the process of rotating the plane table such that plotted lines in the plane table sheet are parallel to the corresponding lines on the ground. This is essential when more than one instrument station is to be used. Orientation is done by (i) Trough compass, (ii) Back sighting.

- **Orientation by trough compass**
  - This is an approximate but quick method of orienting the plane table.
  - At the beginning of surveying, the trough compass is placed on the plane table sheet and the plane table is rotated till the needle becomes central. This is the direction of magnetic north and a line pencil line is ruled against the long side of the box. This represents the North direction.
  - At any later stage of survey, when the plane table requires to be oriented to the north, the compass is placed against this line and the table is turned till the needle freely floats in the middle. The table is then said to be oriented.
- **Orientation by back sighting**
  - This is a more accurate method and two cases rnay arise depending on whether it is possible to set the plane table on a point already plotted on the sheet by way of observation from previous instrument station or not In the first case orientation is done by back sighting.
  - Suppose the line ab has been plotted on the plane table corresponding to the ground line AB. After shifting the plane table from A to B, orientation will be: done as follows:
    a) placing the point b exactly over the Station B with the help of U-frame
    b) alidade is placed along ba and the plane table is rotated such that.station A is sighted from station B through the alidade. When this is achieved, the line ba coincides with the ground line BA and orientation is achieved.

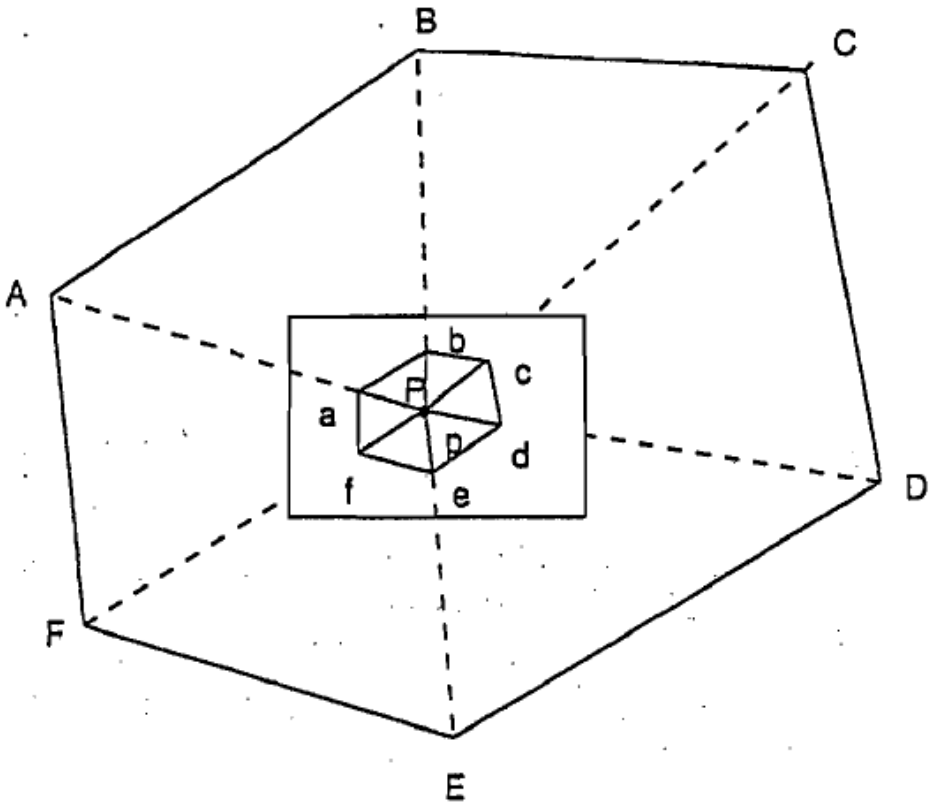
DIFFERENT METHODS OF PLANE TABLE WORK

There are four methods of plane table work:

(i) Radiation. (ii) Intersection (iii) Traversing (iv) Resection.

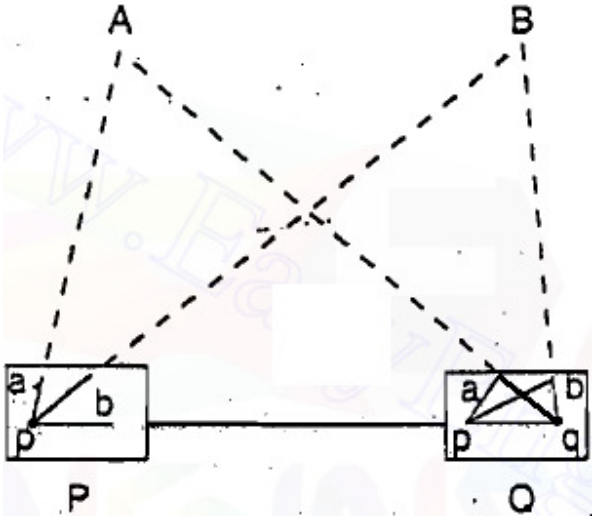
1. RADIATION

- This method is suitable when the ground points are accessible and the distances are small enough to be measured directly using tape.
- Here plane table is placed over station point *P* and all preliminary adjustments performed.
- The alidade is pointed towards *A, B, C, D, E & F* and rays are drawn towards each of these points from point P.
- The lengths PA, PB, PC, PD, PE & PF are measured using tape and points *a, b, c, d, e & f* are plotted on the sheet in proper scale.



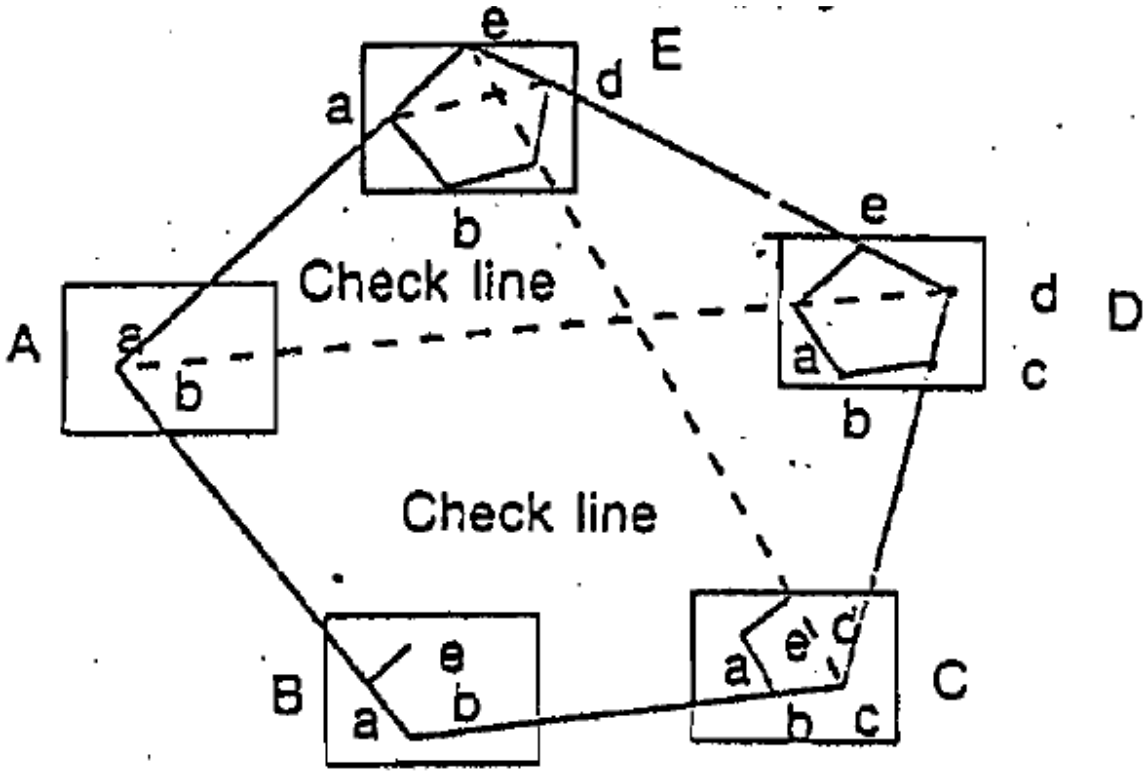
2. INTERSECTION (Graphic Triangulation)

- This method is suitable if the ground points are not accessible from instrument station or the distances are too large.
- Here *P* and *Q* are known instrument stations and the distance PQ is measured
- Plane table is first placed over station *P* and all preliminary adjustments performed.
- From P, alidade is pointed towards *Q* and a ray is drawn and *pq* is plotted on the sheet .
- To locate points *A* and *B,* rays are drawn from *p* towards *A* and *B.*
- The instrument is then shifted to *Q.* The plane table is centred, leveled and properly oriented at *Q* by backsighting A.
- With the alidade pivoted at *q,* rays are drawn toward *A* and *B* on the plane table sheet. The intersection of corresponding two rays defines the position of *A* and *B.*



3. TRAVERSING

- This method is suitable for laying down and plotting survey lines between instrument stations in a closed/open traverse.
- Here, the initial station *A* is occupied and preliminary adjustments performed. From A, point *B* is sighted and AB is measured and the point b is plotted to suitable scale.
- Then instrument is shifted to station *B* and the plane table is centred, leveled and properly oriented at B by backsighting A.
- Pivoting the alidade about b, sight to C and draw a ray. Measure BC and plot c to suitable scale.
- Then instrument is shifted to station *C* and the process is repeated till the final traverse station.



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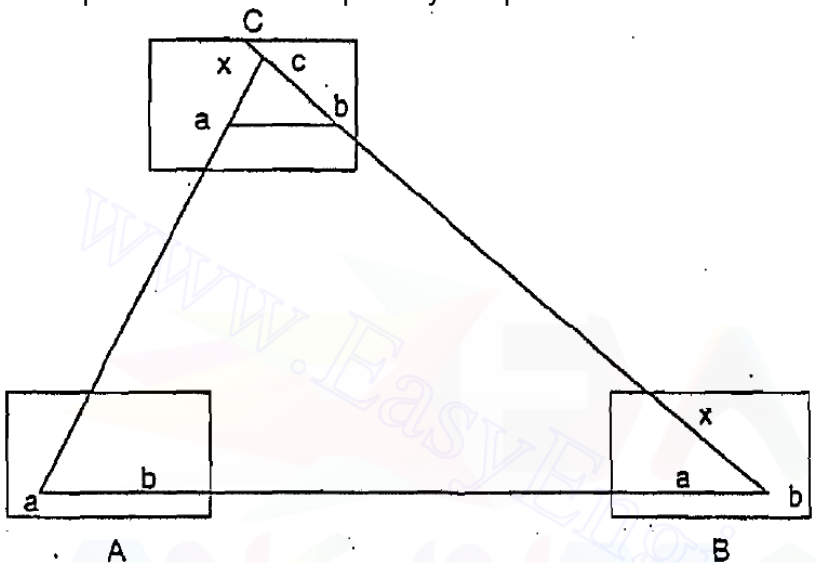
4. RESECTION

- This is the process of determining the ***position of the station occupied by the plane table*** by means of sights taken towards known points, locations of which have already been plotted.
- There are two field conditions: (i) three-point problem (ii) two-point problem
- **Three-point problem:**
    - Here three well-defined points in the field and their corresponding positions in the plane are known.
    - The plane table is placed in an arbitrary position from where the three points are visible. It is necessary to locate the position of the observer. The solution enables the surveyor to place the plane table at any suitable position for taking details.
    - The three-point problem has long been employed in navigation to ascertain ship's position by observing with a sextant on three recognizable features on the shore.
    - There are main methods for solving the three-point problem. They are:
        - Tracing paper method
        - Lehmann's method
        - Analytical method
        - Graphical solution

- **Two point problem:**
    - Here three well-defined points in the field and their corresponding positions in the plane are known.
    - Here two points A and *B* are visible from the instrument station C and the corresponding points *a* and *b* are given in the plane table sheet. Two cases can arise either the points can be occupied by the plane table, or the points cannot be occupied.
    - ***Case I:*** When the known points can be occupied by the plane table



        - Let *a* and *b* be the corresponding points of the ground points *A* and B.
        - The plane table is set up at *B* and oriented by sighting *A.* From *B* a ray *bx* is drawn towards C.

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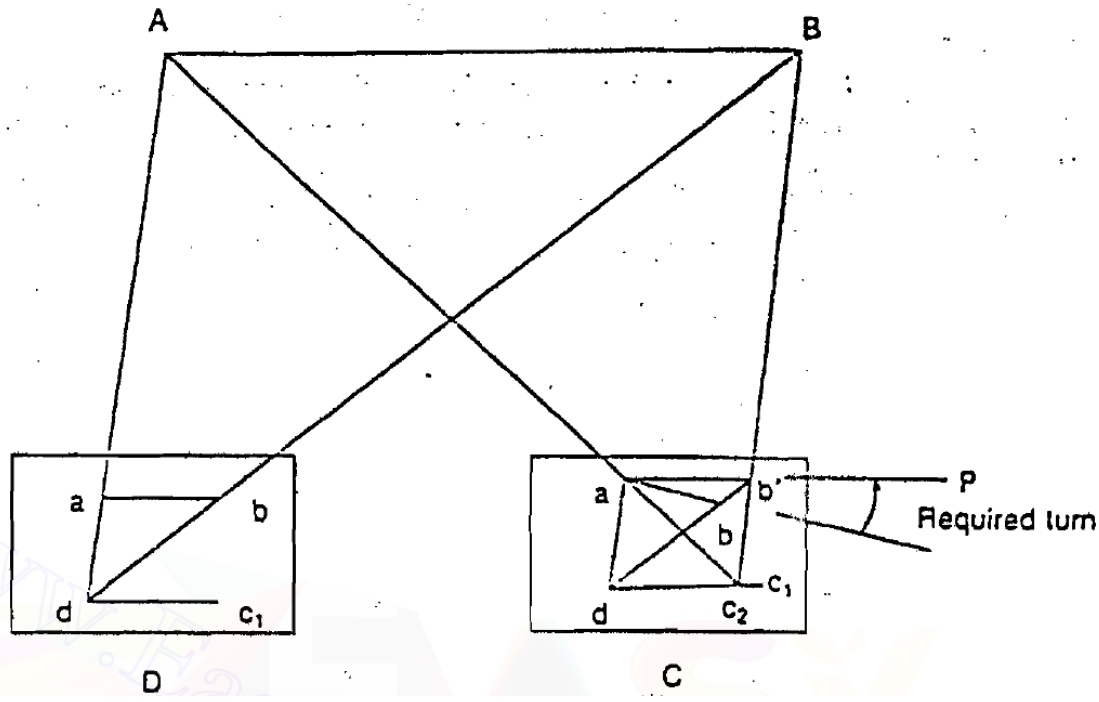
- The table is then shifted to C and oriented by back sighting *B* along *xb* and clamped.
- To locate point C which is on the line *bx,* the alidade is pivoted over *a* and point *A* is sighted. The line *Aa* when produced backward cuts the line *bx* at c to fix the point C

- ***Case II:*** When the plane table cannot be placed over the known controlling stations
  - An auxilliary station point *D* is to be chosen near C. Set the table at *D* in such a way that *ab* is approximately parallel to *AB.* Clamp the table.
  - With alidade pivoted at *a,* sight *A* and draw a line. Similarly with alidade at *b* sight *B* and draw a ray. The two rays intersect at point *d.* Transfer this point to the ground. This is point D on the ground.
  - Now, pivot the alidade at *d* and draw a ray sighting C. Measure DC using tape and mark c_1.
  - Shift the table to C and back sight D with reference c_1d to orient the plane table
  - With alidade pivoted at *a,* sight *A.* This ray intersects the previously drawn ray c_1d at c_2.
  - From c_2, sight B and draw a ray to intersect the ray *db* produced at *b',* Thus *b'* is the approximate location of *B* with respect to the orientation made at D.
  - The angle between *ab* and *ab'* is the error in orientation. The board should be rotated. through the angle *bab',* This is' done by fixing a pole at *P* such that it is in line with *ab'.* The plane table is then rotated till *ab* comes in line with *P.* The table is thus correctly oriented.
  - From this new position, draw rays *Aa* and *Bb,* They will intersect at *c* (not shown here), which will be the point corresponding to *C.*



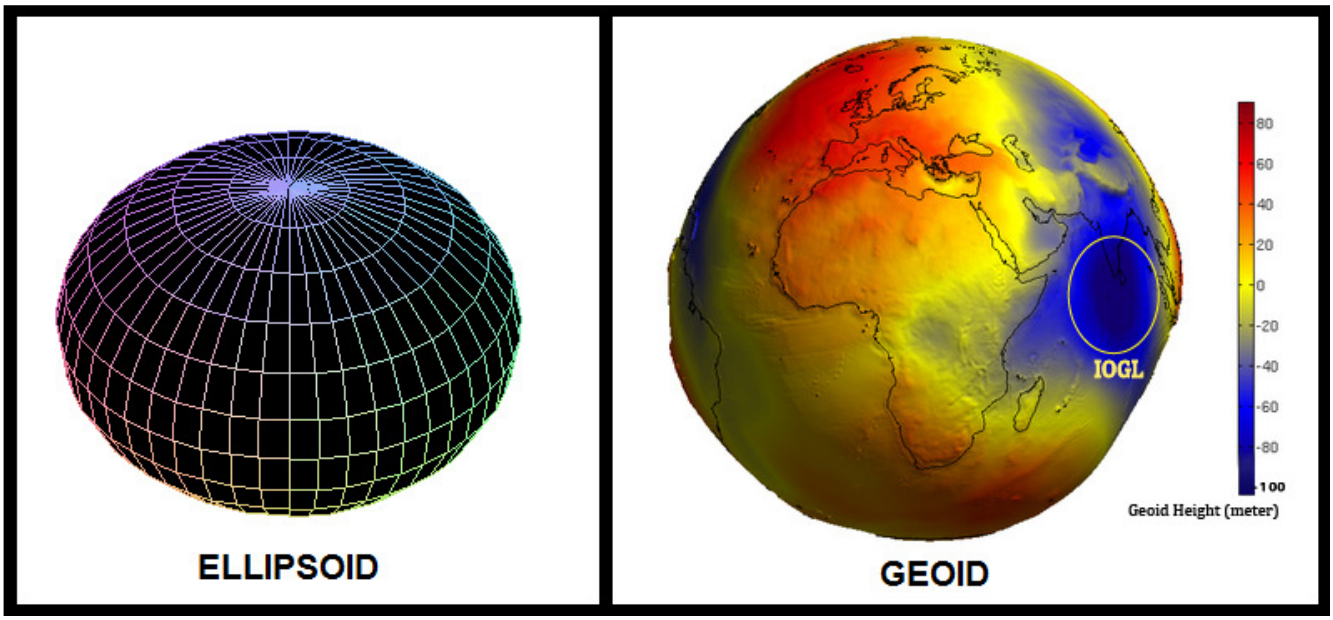
CHAPTER 4

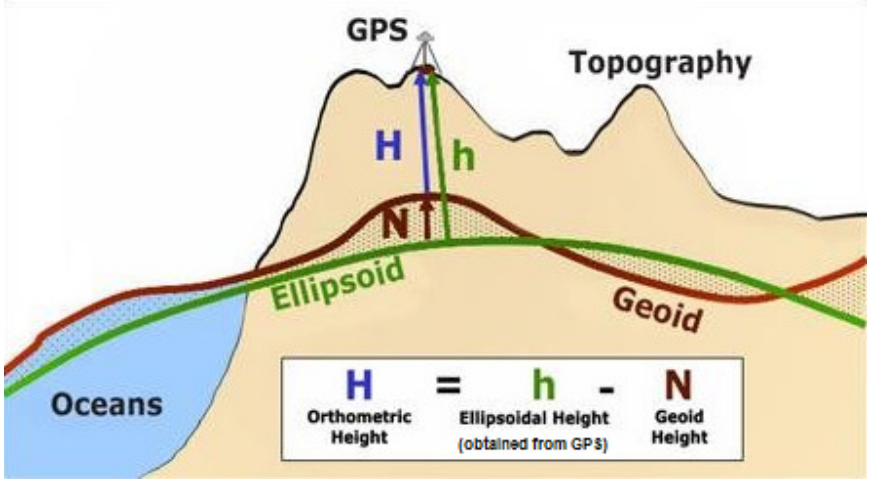
LEVELLING

Leveling is the general term applied to any of the various processes by which elevations of points or differences in elevation are determined.

IMPORTANT TERMINOLOGIES

1. **Vertical line:** A line that follows the local direction of gravity as indicated by a plumb line.
2. **Level surface:** A curved surface that is perpendicular to the local plumb line.
   - Level surfaces are approximately *spheroidal* or *ellipsoidal* in shape.
   - The free surface level of a body of still water is the best example of a level surface.
   - Within local areas, level surfaces at different heights are considered to be concentric.
3. **Level line:** A line in a level surface—therefore, a curved line.
4. **Horizontal plane**: A plane surface perpendicular to the local direction of gravity. In plane surveying, it is a plane perpendicular to the local vertical line.
5. **Horizontal line:** A line in a horizontal plane.
6. **Datum or Vertical datum or Reference Datum:** Any level surface to which elevations are referenced. This is the surface that is arbitrarily assigned zero elevation.
7. **Elevation:** The distance measured along a vertical line from a vertical datum to a point or object. The elevation of a point is also called *orthometric height*.
8. **Geoid:** It is the shape that the ocean surface would take under the influence of the gravity and rotation of Earth only, without the effect of winds and tides. It serves as a datum for all elevations and astronomical observations.
9. **Ellipsoid:** It is the mathematical figure approximating the form or shape of Earth.

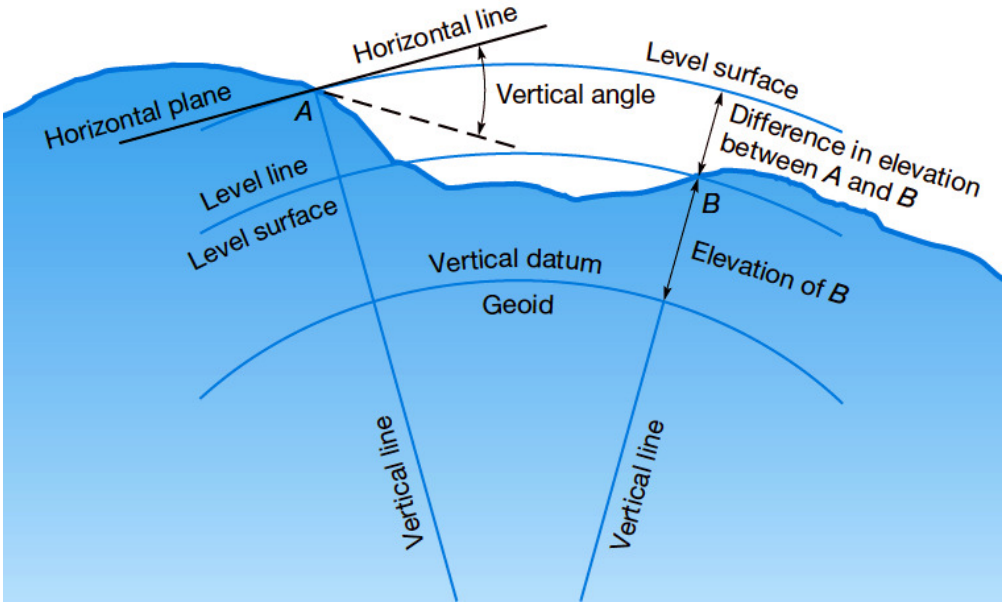




10. Mean sea level (MSL): MSL was defined as the average height for the surface of the seas for all stages of tide over a 19-year period. The current Indian vertical datum was defined during the period 1905-1909 on the basis of sea level observations at nine tidal observatories (four each on east and west coast and one in Karachi).

11. Benchmark (BM): A relatively permanent object, natural or artificial, having a marked point whose elevation above or below a reference datum is known or assumed.

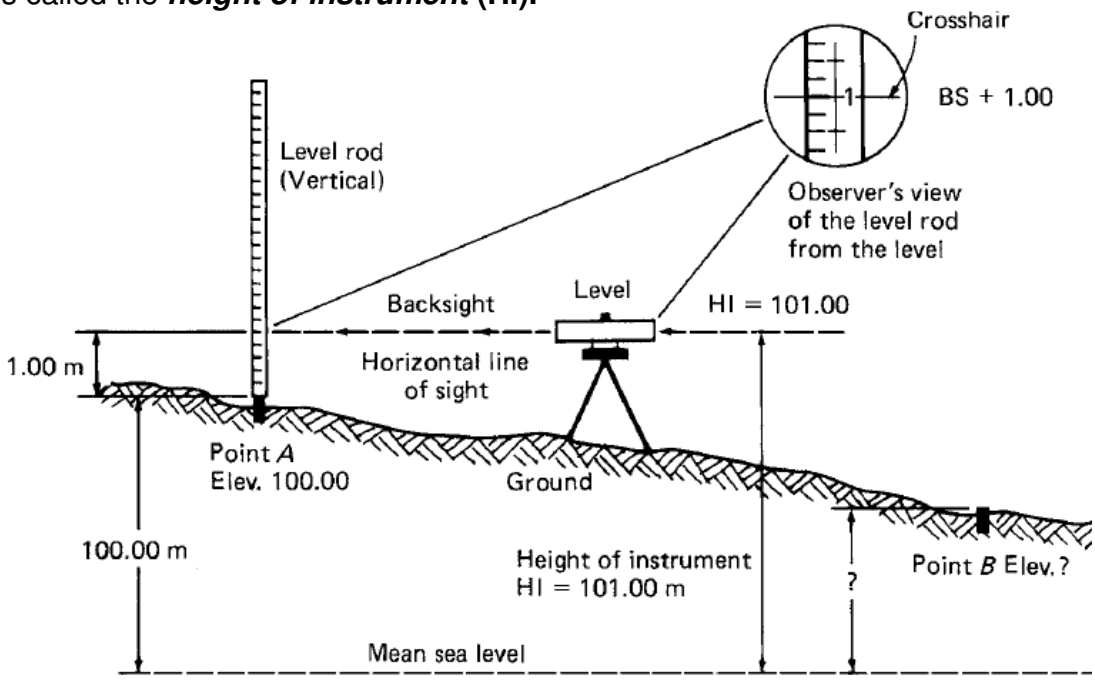
12. Vertical control: A series of benchmarks or other points of known elevation established throughout an area, also termed basic control or level control.



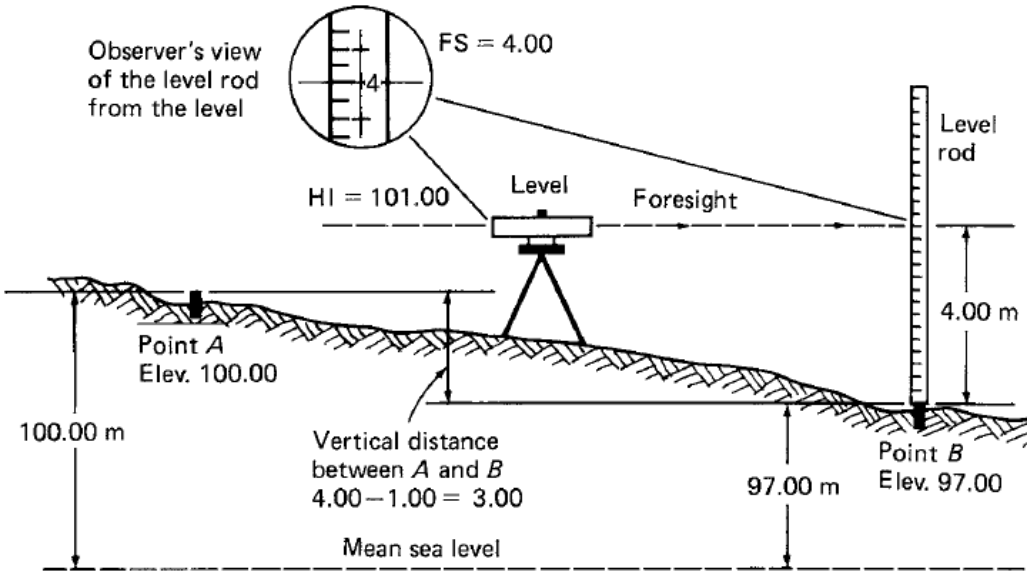
THEORY OF LEVELLING

- A horizontal line of sight is first established with an instrument called a ***level***. The level is securely mounted on a stand called a *tripod*, and the line of sight is made horizontal.
- Then the surveyor looks through the telescopic sight toward a graduated *level rod* ***(leveling staff)***, which is held vertically at a specific location or point A on the ground (see Figure *a*).

- A reading is observed on the rod where it appears to be intercepted by the horizontal cross hair of the level; this is the vertical distance from the point on the ground up to the line of sight of the instrument. Generally, the elevation of point *A* is already known; otherwise, it is assumed. The rod reading on a point of known elevation is termed a ***backsight*** **(BS)** reading. It is also often called a ***plus sight*** (+S) reading because it generally must be added to the known elevation of point *A* to determine the elevation of the line of sight.
- For example, suppose the elevation of point *A* is 100.00 m (above MSL), and the rod reading is 1.00 m. From Figure *a*, it is clear that the elevation of the line of sight is equal to 100.00 + 1.00 = 101.00 m. The elevation of the horizontal line of sight through the level is called the ***height of instrument*** **(HI).**



(*a*)



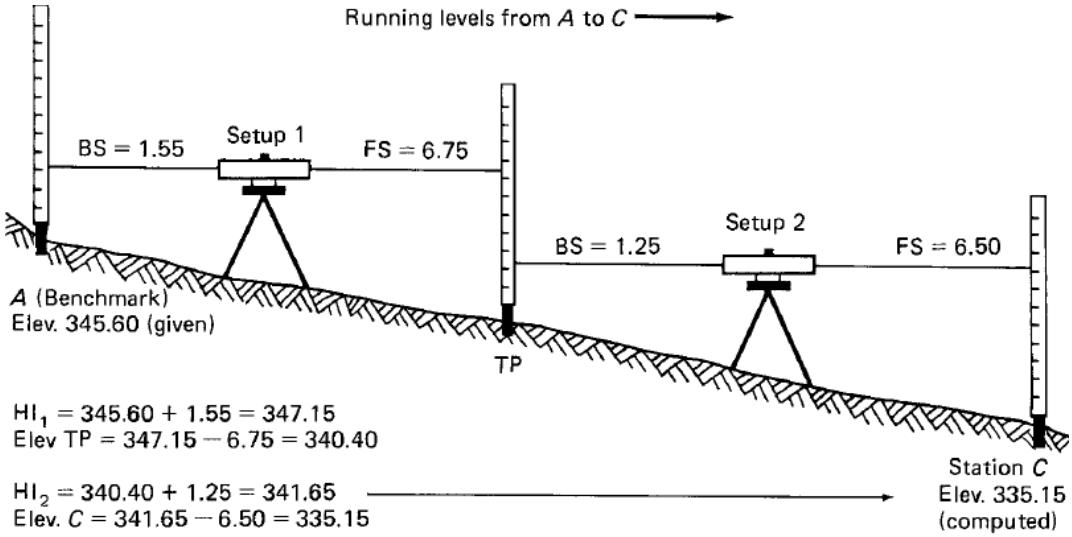
(*b*)

- Suppose we must determine the elevation of point *B* (see Figure *b*). The *instrument person* (the surveyor operating the level) turns the telescope so that it faces point *B* and reads the rod now held vertically on that point. For example, the rod reading might be 4.00 m. A rod reading on a point of unknown elevation is called a ***foresight* (FS)**, or a ***minus sight*** (-S). Because the HI was not changed by turning the level, we can simply subtract the foresight reading of 4.00 from the HI of 101.00 to obtain the elevation of point *B,* resulting here in 101.00 - 4.00 = 97.00 m.
- The process of determining the level difference between two nearby points on the ground is called ***differential leveling***. The difference between the two rod readings is, in effect, the vertical distance between the two points. In the preceding example, the vertical distance between *A* and *B* may be computed as either 100.00 - 97.00 = 3.00 m, or 4.00 - 1.00 = 3.00 m. Although it is the vertical distance that is actually being measured, the results are generally expressed as the elevations of points above a common reference plane or datum.
- This basic cycle of differential leveling can be summarized as follows:

$$\begin{aligned}\text{Height of instrument} &= \text{known elevation} \\ &\quad + \text{backsight or HI} \\ &= \text{Elev}_A + \text{BS}\end{aligned}$$

$$\begin{aligned}\text{New elevation} &= \text{height of instrument} \\ &\quad - \text{foresight or Elev}_B \\ &= \text{HI} - \text{FS}\end{aligned}$$

- Often, the elevations of several points over a relatively long distance must be determined. A process called ***running levels (or fly leveling)*** is used to determine the elevations of two or more widely separated points. It simply involves several cycles or repetitions of the basic differential leveling operation.
- Suppose it is necessary to determine the elevation of point *C* relative to point *A* (see Figure below). But in this case, let us assume that it is not possible to set up the level so that both points *A* and *C* are visible from one position (due to either physical obstacles or excessive distance).



- The line of levels can be carried forward toward *C* by establishing a convenient and temporary ***change point (CP)*** or ***turning point*** **(TP)** somewhere between *A* and *C.* The selected TP serves merely as an intermediate reference point; it does not have to be actually set in the ground as a permanent monument.
- The elevation of the turning point is computed from the first pair of BS and FS readings. The BS is on point *A,* which is the point of known elevation. A secure and permanent point of known elevation is called a ***benchmark*** **(BM)**; a leveling survey should always begin with a backsight on a benchmark, such as benchmark *A* (BM_A). The BS is added to the elevation to give the HI at the first instrument position. The elevation of the turning point is obtained by subtracting the FS from the HI. Once the elevation of the turning point is known, the level instrument can be moved to another location, one closer to *C* but still in sight of the turning point. Then another backsight is taken, this time on the turning point, to determine the new height of instrument. Finally, a foresight is taken on point *C*, and its elevation is computed.

- Computations for leveling simply involve successive additions and subtractions. But when running levels with several turning points (and twice as many rod readings), it is necessary to keep a well-organized field book to keep track of which numbers are added or subtracted from which. The computations are hence set up in a typical field book format, based on any of the two methods.

    - **Height of collimation or Height of Instrument method:**
        - In this system,the height of the line of collimation is found by adding the backsight reading to RL of the BM on which the BS is taken.
        - Then the RL of the intermediate points and the change point are obtained by subtracting the respective staff readings from the height of Instrument (HI).
        - The level is then shifted for the next set up and again the height of the line of collimation is obtained by adding the backsight reading to the RL of the change point(which is calculated in the first setup).
        - So the height of instrument is different in different set ups of the level. Two adjacent planes of collimation are correlated at the change point by an FS reading from one setting and a BS reading from the next setting.
        - The RLs of of unknown points are to be found out by deducting the staff readings from the RL of the height of instrument.
        - Arithmetic check: ΣBS – ΣFS = Last RL – First RL

Station point	Chainage	BS	IS	FS	RL of Line of collimation(HI)	RL	Remarks
1	165	3.150	xxxx	xxxx	101.235	98.085	
2	180	xxxx	2.245	xxxx	xxxx	98.990	
3	195	xxxx	1.125	xxxx	xxxx	100.110	
4	210	3.125	xxxx	0.860	103.500	100.375	Change Point
5	225	xxxx	2.760	xxxx	xxxx	100.740	
6	240	xxxx	1.835	xxxx	xxxx	101.665	
7	255	xxxx	1.470	xxxx	xxxx	102.030	
8	270	1.225	xxxx	1.965	102.760	101.535	Change Point
9	285	xxxx	2.390	xxxx	xxxx	100.370	
10	300	xxxx	xxxx	3.035	xxxx	99.725	
TOTAL=		**7.500**	xxxx	**5.860**	xxxx	xxxx	

Arithmetic Check: $\sum BS - \sum FS$ =Last RL -1st RL,(7.500-5.860)=+1.640,(99.725-99.085)= +1.640

- **Rise and Fall method:**
  - In this method, the difference in level between two consecutive points is determined by comparing each forward staff reading with the staff reading at the immediately preceding point.
  - If the forward staff reading is smaller than the immediately preceding staff reading, a rise is said to have occurred. The rise is added to the RL of the preceding point to get the RL of the forward point.
  - If the forward staff reading is greater than the immediately preceding staff reading, it means there has been a fall. The fall is subtracted from the RL of the preceding point to get the RL of the forward point.
  - Arithmetic check: ΣBS – ΣFS = ΣRise – ΣFall = Last RL – First RL

Station point	Chainage	BS	IS	FS	Rise(+)	Fall(-)	RL	Remarks
1	165	3.150	xxxx	xxxx	xxxx	xxxx	98.085	
2	180	xxxx	2.245	xxxx	0.905	xxxx	98.990	
3	195	xxxx	1.125	xxxx	1.120	xxxx	100.110	
4	210	3.125	xxxx	0.860	0.265	xxxx	100.375	ChangePoint
5	225	xxxx	2.760	xxxx	0.365	xxxx	100.740	
6	240	xxxx	1.835	xxxx	0.925	xxxx	101.665	
7	255	xxxx	1.470	xxxx	0.365	xxxx	102.030	
8	270	1.225	xxxx	1.965	xxxx	0.495	101.535	ChangePoint
9	285	xxxx	2.390	xxxx	xxxx	1.165	100.370	
10	300	xxxx	xxxx	3.035	xxxx	0.645	99.725	
TOTAL=		**7.500**	**xxxx**	**5.860**	**3.945**	**2.305**	**xxxx**	

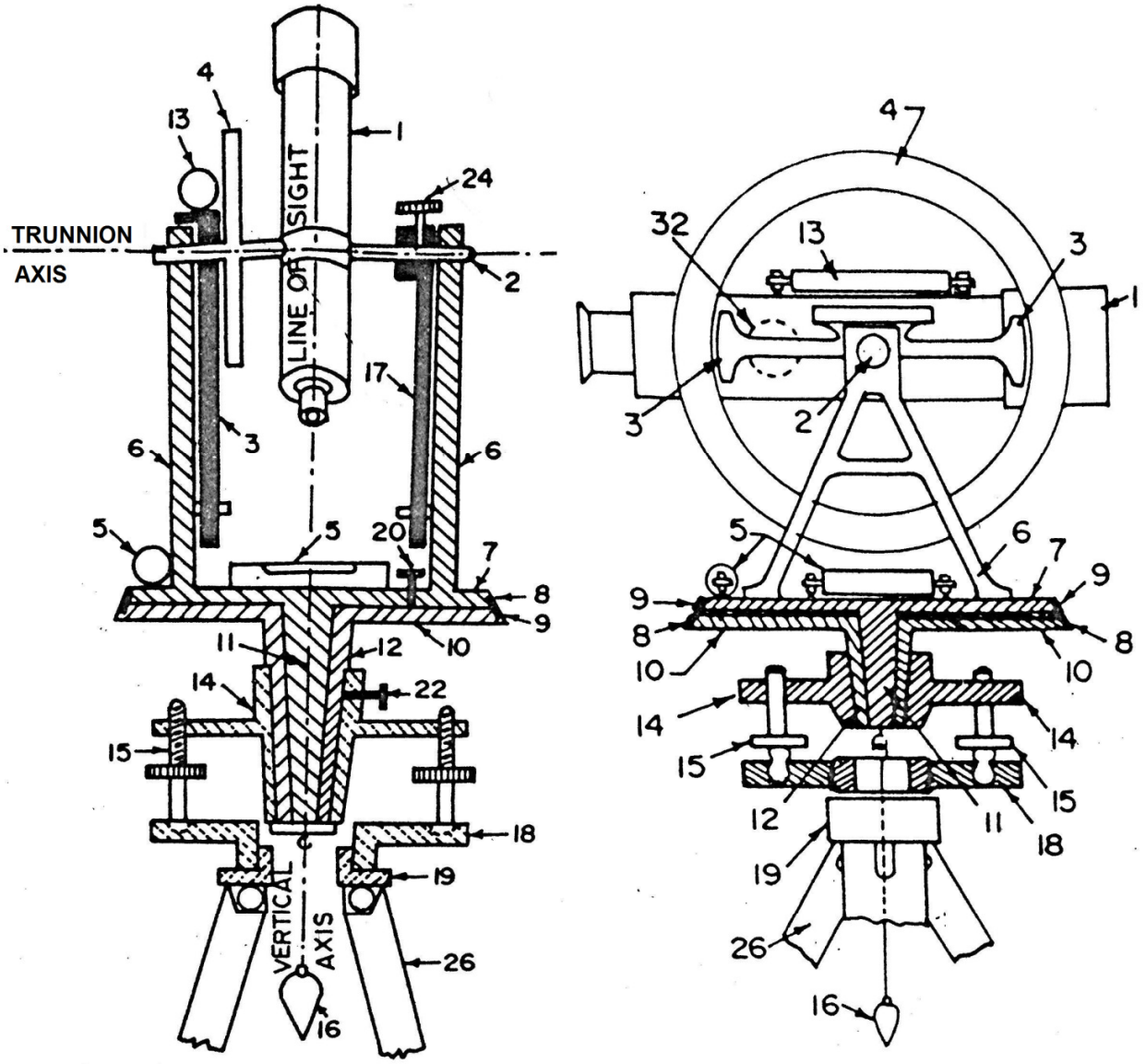
$\sum BS - \sum FS$ =7.500-5.860=+1.640, Last RL -1st RL=(99.725-98.085=+1.640,

$\sum Rise - \sum Fall$ =3.945-2.305=+1.640

CHAPTER 5

STUDY OF THEODOLITE

- **Theodolite** - a precise optical instrument
  - used to measure horizontal and vertical angles



1. Telescope
2. Trunnion Axis
3. Vernier Frame
4. Vertical Circle
5. Plate Levels
6. Standards (A-Frame)
7. Upper Plate
8. Horizontal Plate vernier
9. Horizontal Circle
10. Lower Plate
11. Inner Axis
12. Outer Axis
13. Altitude Level
14. Levelling Head
15. Levelling Screw
16. Plumb Bob
17. Arm Of Vertical Circle Clamp.
18. Foot Plate
19. Tripod Head
20. Upper Clamp
22. Lower Clamp
24. Vertical Circle Clamp
26. Tripod
32. Focusing screw

CLASSIFICATION OF THEODOLITES

I. PRIMARY CLASSIFICATION:

1. **Transit Theodolite** – Telescope can be rotated about horizontal axis in the vertical plane

   Eg: Everest Theodolite, Wild T2 Theodolite etc.

2. **Non-transit theodolites** - Telescope cannot be rotated about horizontal axis

   Eg: Plain Theodolite, Y- Theodolite etc.

II. SECONDARY CLASSIFICATION:

1. **Vernier Theodolite** – Verniers fitted to read graduated circles for angular measurements

   Least count – 20”

2. **Micrometer Theodolite** – Micrometer microscopes fitted to read graduated circles

   Least count – 1”

PARTS OF A TRANSIT THEODOLITE

1. Telescope: It consists of eye-piece, object glass and focusing screw and it is used to sight the object. Mounted on horizontal axis (trunnion axis).

2. Vertical Circle: rigidly attached to telescope; has graduations from 0-360^o or divided into 4 quadrants (measuring 0 – 90^o). It is used to measure vertical angles. The line joining zeros is parallel to the line of sight.

3. A-Frame (Standards): supports trunnion axis of the telescope; Also supports T-frame (or Index Frame) and vertical circle clamp.

4. Index frame (T-Frame): supports the vernier on the vertical circle. It carries the level tube called “Altitude Bubble”.

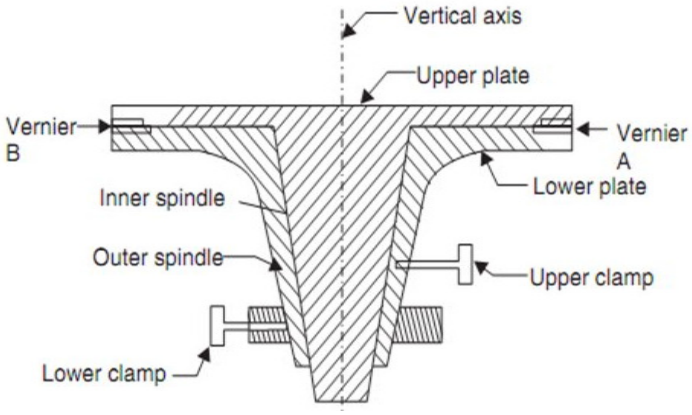
5. Levelling Head: It consists of two parallel triangular plates (called tribrach plates) & three footscrews. Its uses are

a. To support the main part of the instrument.

b. To attach the theodolite to the tripod.

c. To level the instrument.

6. **Spindles:** conical arrangements to which upper & lower plates are separately fixed.



Inner spindle – carries upper plate (carries verniers)

Outer spindle – carries lower plate (carries main circular scale)

7. **Lower Plate:**

- Attached to outer spindle
- Carries main circular scale for horizontal angle measurements. Therefore, lower plate is also called Scale plate
- It consists of lower clamp screw (for fixing) and tangent screw (for fine adjustment)

8. **Upper Plate:**

- carries two vernier scales at diametrically opposite points.
- Supports A-frame
- consists an upper clamp screw and tangent screws

9. **Foot Screws:** These are used to level the instrument

10. **Plate Levels:** fixed to upper plate; placed parallel to trunnion axis.

11. **Plumb Bob:** It is used to center theodolite exactly over the ground station mark.

TECHNICAL TERMS

1. **Swinging The Telescope:** It means rotating the telescope about its vertical axis in the horizontal plane. A swing is called right or left according as the telescope is rotated clockwise or counter clockwise.

- If the telescope is rotated clockwise => Right swing
- If the telescope is rotated anticlockwise => Left swing

2. **Transiting the theodolite:** Rotating the telescope in the vertical plane, through 180°. Since Line of sight is reversed in this operation, it is also called “Reversing” or “Plunging”.

3. **Horizontal axis (Trunnion axis):** axis about which the telescope transits.

4. **Vertical axis:** axis about which upper & lower plates rotate.

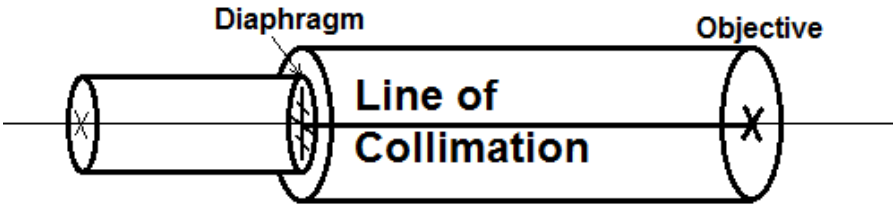
5. **Face Left:** If face of the vertical circle is to the left side of the observer, then the observation of the angles taken is known as face left observation.

6. **Face Right:** If the face of the vertical circle is to the right side of the observation, then the observation of the angles taken is known as face right observation.

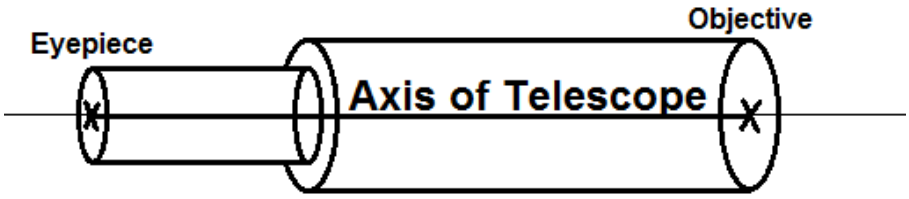
7. **Changing Face:** It is an operation of bringing the face of the telescope from left to right and vice-versa.

8. **Axis of level tube (Bubble line):** Line drawn tangential to the longitudinal curve of the level tube, at the centre of the bubble. If the bubble is central, then the axis of the level tube is horizontal.

9. **Line of Sight (LOS) or Line of Collimation (LOC):** passes through the optic center of objective lens and intersection of crosshairs of the diaphragm.



10. **Axis of the Telescope:** It is also known an imaginary line joining the optical center of the objective lens to the optical center of the eyepiece.



11. **Telescope normal:** when

- the face of the vertical circle is to the left of observer and
- sighting vane or the bubble of the telescope is above telescope

12. **Telescope inverted:** when

- the face of the vertical circle is to the right of observer and
- sighting vane or the bubble of the telescope is below telescope

TEMPORARY ADJUSTMENTS OF THEODOLITE:

There are three temporary adjustments of a theodolite. These are

1. Setting up the theodolite over a station.

2. Leveling up.

3. Elimination of parallax.

1. Setting Up:

It includes two operations

1. Centering a theodolite over a station: Done by means of plumb bob.

2. Approximately leveling it by tripod legs only: Done by moving tripod legs radially or circumferentially.

2. Leveling Up:

Having centered and approximately leveled the instrument, accurate leveling is done with the help of foot screws with reference to the plate levels, so that the vertical axis shall be truly vertical. To level the instrument the following operations have to be done.

1. Turn the upper plate until the longitudinal axis of the plate level is roughly parallel to a line joining any two of the leveling screws (A & B).

2. Hold these two leveling screws between the thumb and first finger of each hand uniformly so that the thumb moves either towards each other or away from each other until the bubble comes to the center.

3. Turn the upper plate through 90º i.e until the axes of the level passes over the position of the third leveling screw 'C'.

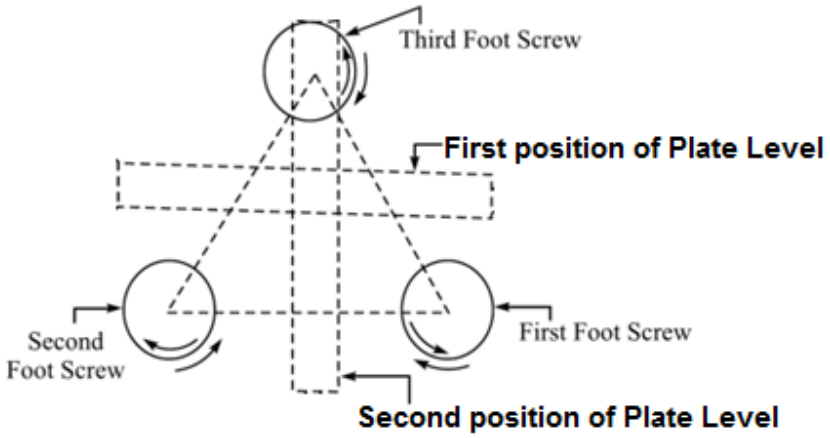
4. Turn this leveling screw until the bubble comes to the center.

5. Rotate the upper plate through 90º to its original position fig(a) and repeat step(2) till the bubble comes to the center.

6. Turn back again through 90º and repeat step 4.

7. Repeat the steps 2 and 4 till the bubble is central in both the positions.

8. Now rotate the instrument through 180º. The bubble should be remaining in the center of its run, provided it is in correct adjustment. The vertical axis will then be truly vertical.



3. Elimination Of Parallax:

Parallax is a condition arising when the image formed by the objective is not in the plane of the cross hairs. Unless parallax is eliminated, accurate sighting is not possible. Parallax can be eliminated in two steps.

A. ***Focussing the Eye-Piece:*** Point the telescope to the sky or hold a piece of white paper in front of the telescope. Move the eyepiece in and out until a distant and sharp black image of the cross-hairs is seen.

B. ***Focussing the Object:*** Telescope is now turned towards object to be sighted and the focusing screw is turned until image appears clear and sharp.

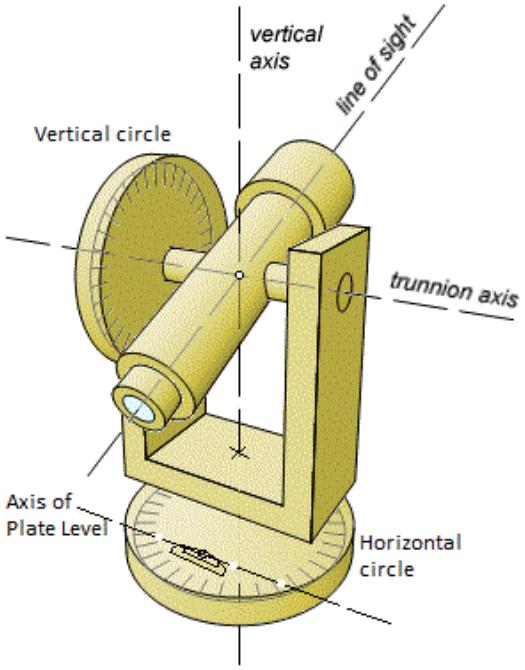
PERMANENT ADJUSTMENTS OF THEODOLITE

Fundamental lines of Theodolite:

1) Vertical axis
2) Horizontal axis (or Trunnion axis)
3) Line of Sight (or Line of collimation)
4) Axis of plate level
5) Axis of altitude level

Relationship between Fundamental Lines of Theodolite:

1) **Axis of plate level** must be perpendicular to **Vertical axis**.
2) **Line of Sight** must be perpendicular to **Horizontal axis.**
3) **Horizontal axis** must be perpendicular to **Vertical axis.**
4) **Axis of altitude level** must be parallel to **Line of Sight.**
5) **Vertical circle** reading must be zero if **Line of Sight** is horizontal.



Permanent adjustments of Theodolite:

Permanent adjustments involve setting the essential parts of a Theodolite into their true positions relatively to each other. This is ensured as long as the relationship between fundamental lines of the Theodolite hold good.

- The permanent adjustments of a Theodolite are in the following order.
    1) Adjustment of plate level
    2) Adjustment of line of sight
    3) Adjustment of the horizontal axis
    4) Adjustment of altitude bubble and vertical index frame
- The object of the permanent adjustments are as follows:
    1) ***Adjustment of plate level*** – to make vertical axis truly vertical. Hence, if the instrument is leveled up, the bubble of plate level becomes central.
    2) ***Adjustment of line of sight*** – to place the intersecting point of horizontal and vertical crosshairs in the optical axis of the telescope. Thus it involves adjustments of both horizontal & vertical crosshairs.
    3) ***Adjustment of the horizontal axis*** – to make horizontal axis perpendicular to the vertical axis. Hence, the horizontal axis is truly horizontal when the instrument is leveled.
    4) ***Adjustment of altitude bubble and vertical index frame*** – to make Line of Sight truly horizontal when the altitude bubble is central & vertical circle reading is zero.

MEASUREMENT OF HORIZONTAL ANGLES

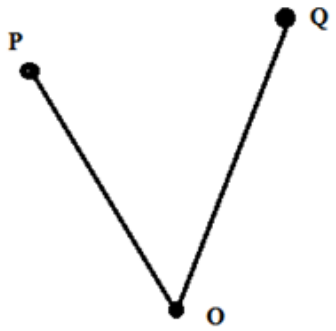
To measure the horizontal angle PQR,

- Set up the instrument at Q and perform all temporary adjustments.
- Release all clamps. Turn the upper and lower plates in opposite directions till the zero of one of the vernier (say A) is against the zero of the scale and the vertical circle is to the left.
- Clamp both the plates together by upper clamp and lower clamp and bring the two zeros into exact coincidence by turning the upper tangent screw. Take both vernier readings.
- The reading on vernier B will be 180 degree, if there is no instrumental error.
- Set the telescope in Face Left position.
- Loose the lower clamp and turn the instrument towards point P. Bisect point P accurately by using lower tangent screw. Check the readings of verniers A and B. There should be no change in the previous reading.

- Unclamp the upper clamp and rotate the instrument clockwise about the inner axis to bisect the point R. Clamp the upper clamp and bisect R accurately by using upper tangent screw. Read both verniers. The reading of vernier A gives the angle PQR directly while the vernier B gives by deducting 180 degree.
- While entering the reading, the full reading of vernier A (i.e., degrees, minutes and seconds) should be entered, while only minutes and seconds of the vernier B are entered.
- The mean of the two such vernier readings gives the angle PQR with one face. Change the face by transiting the telescope and repeat the whole process.

Horizontal angle is measured by any of the following two methods.

1) Repetition Method



- Set up the instrument over 'O' and level it accurately.
- With the help of upper clamp and tangent screw, set 0º reading on vernier 'A'. Note the reading of vernier 'B'.
- Release the upper clamp and direct the telescope approximately towards the point 'P'. Tighten the lower clamp and bisect point 'P' accurately by lower tangent screw.
- Release the upper clamp and turn the instrument clock-wise towards Q. Clamp the upper clamp and bisect 'Q' accurately with the upper tangent screw. Note the readings of verniers 'A' and 'B' to get the values of the angle POQ.
- Release the lower clamp and turn the telescope clockwise to sight P again. Bisect P by using the lower tangent screw.
- Release the upper clamp, turn the telescope clockwise and sight Q. Bisect Q by using the upper tangent screw.
- Repeat the process until the angle measured (required number of times is 3). The average angle with face left will be equal to final reading divided by three.
- Change face and make three more repetitions as described above. Find the average angle with face right, by dividing the final reading by three.
- The average horizontal angle is then obtained by taking the average of the two angles with face left and face right.

Errors eliminated by Repetition Method:

- ⊙ Errors due to eccentricity of verniers are eliminated by taking both vernier readings.
- ⊙ Errors due to inadjustments of line of collimation and trunnion axis are eliminated by taking both face readings.

- ⊙ Errors due to inaccurate graduations in horizontal circle are also eliminated by taking readings at different parts of the circle.
- ⊙ Errors due to inaccurate bisection of object, eccentric centring etc. are eliminated due to multiple sightings of objects.

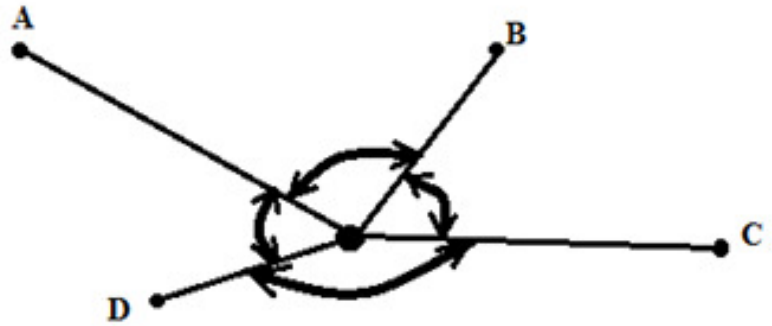
Errors which cannot be eliminated by this method:

- ⊙ Errors due to non-verticality of vertical axis.
- ⊙ Errors due to slip and displacement of station signals.

Limit of precision of this method:

- ⊙ By repeating angular measurements, the operations like sighting, clamping etc. are multiplied and hence opportunities for error are multiplied.
- ⊙ Maximum precision is achieved after the 5^{th} or 6^{th} repetition.

2) Reiteration Method (Direction Method)



- Set the instrument over "O" and level it. Now set the Vernier to zero and bisect point A accurately.
- Loose the upper clamp and turn the Telescope clockwise to point B. Bisect B by using the upper tangent screw. Read both the Verniers, the mean of the Verniers will give the angles AOB.
- Similarly, bisect successively C, D etc, thus closing the circle. Read both the Verniers at each bisection.
- Finally sight to A the reading of the vernier should be the same as the original setting reading. Repeat the steps 02 to 04 with other face i.e. Face Right. The average of angles measured with Face Left & Face Right is then computed.

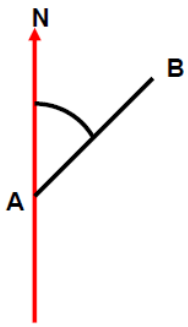
Errors eliminated by Reiteration Method:

- ⊙ Errors due to eccentricity of verniers are eliminated by taking both vernier readings.
- ⊙ Errors due to inadjustments of line of collimation and trunnion axis are eliminated by taking both face readings.
- ⊙ Errors due to inaccurate graduations in horizontal circle are also eliminated by taking readings at different parts of the circle.
- ⊙ Eccentricity of vertical axis is also eliminated.

MISCELLANEOUS OPERATIONS USING THEODOLITE

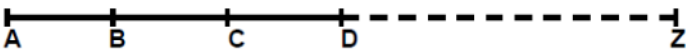
1) To measure Magnetic Bearing of line

- Setup the instrument over A and level it accurately. The Theodolite shall be provided with a tubular compass or trough compass.
- Set the vernier to the zero of the horizontal circle.
- Release the magnetic needle and loosen the lower clamp. Rotate the instrument till magnetic needle points to North.
- Clamp the lower clamp with the help of lower tangent screw. Bring the needle exactly against the mark in order to bring it in magnetic meridian. At this stage the line of sight will be along the magnetic meridian.
- Now loose the upper clamp and point the telescope towards B. With the help of upper tangent screw, bisect B accurately and read both the vernier. The mean of the two readings will be recorded as magnetic bearing of line. Change the face of the instrument for accurate magnetic bearing of the line and repeat.
- The mean of the two values will give the correct bearing of the line AB.



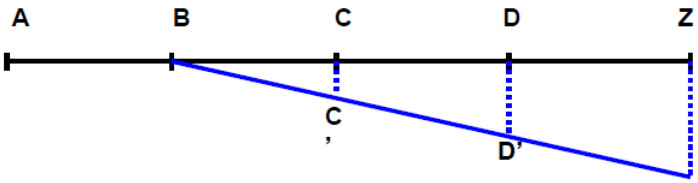
2) To prolong a straight line

- ***1st Method:*** Set the instrument at A and sight B accurately. Establish a point C in the line of sight AB produced. Now shift the instrument to B and sight C. Establish the point D along the line of sight BC produced. Repeat the process.



- ***2nd Method*:** Set the instrument at B; backsight A; transit the telescope and establish a point C in the line of sight. Similarly shift the instrument to C and backsight B; transit the Theodolite to establish point D. Repeat the process.

  Note: If the instrument is in permanent adjustment, points B, C, D, …… will be in the straight line. Otherwise, the points established shall be C', D',….. which shall not be in straight line.



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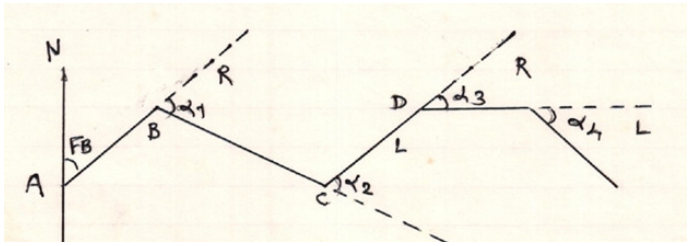
3) To measure deflection angles



- Deflection angle = angle which a survey line makes with the prolongation of the preceding line. Range from 0-180^{o}.
- It is designated as Left(L) or Right(R), depending on whether the angle is measured anti-clockwise or clockwise.
- Here, the prolongation of line is performed as above and the horizontal angle is measured from this to the adjacent surveyline.

CHAPTER 6

THEODOLITE TRAVERSE

Traversing refers to the survey which involves a framework of survey lines whose length and direction are measured. If this is performed using a Theodolite, it is called Theodolite Traverse.

Traverse can be of 2 types:

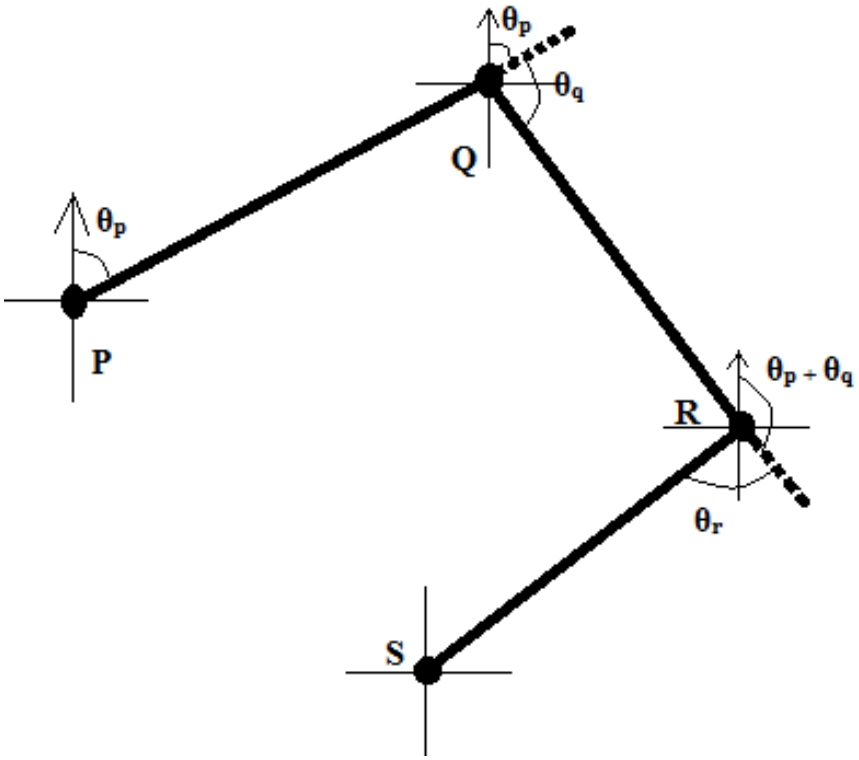
1) ***Closed Traverse:*** if the circuit of surveylines begins and ends at the same point.
   Eg: survey of large area, locating boundaries of lakes, forests etc.
2) ***Open Traverse:*** if the circuit of survey lines does not begin and end at the same point.
   Eg: survey of roads, railways, canals etc.

DIFFERENT METHODS OF THEODOLITE TRAVERSE OR TRANSIT-TAPE TRAVERSE is given below:

- Fast Needle method – involves measurement of magnetic bearings
- Direct measurement of angles between lines – involves measurement of angles

Fast Needle Method

- Requires a Theodolite fitted with a tubular compass
- Involves direct measurement of magnetic bearing of surveyline at the first station only. Magnetic bearings of other lines are measured with respect to the bearing at the first station.
- There are 3 methods of observing bearings of survey lines:
  1) ***Direct Method with Transiting***



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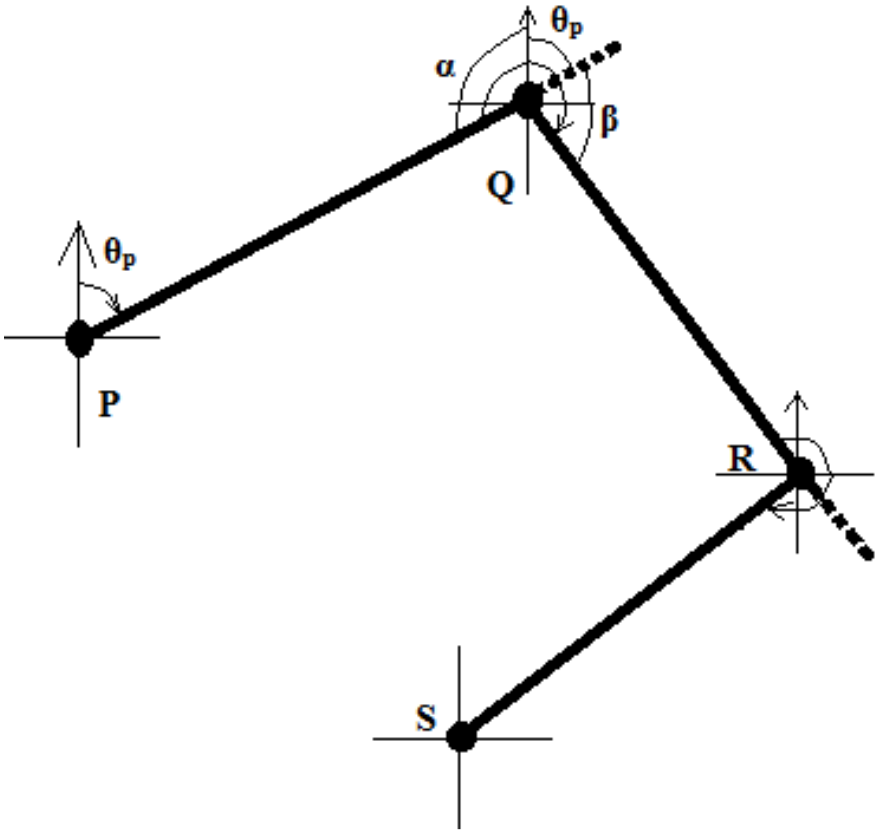
- Let there be points P, Q, R, S, etc. on the field along which the traverse is required to be performed.
- Set the Theodolite at P and perform all temporary adjustments. Also set the horizontal circle reading to zero. Using the lower clamp and tangent screws, orient the line of sight to Magnetic Meridian (or North).
- Loosen the upper clamp screw and swing the telescope rightwards to sight point Q and bisect it exactly. Horizontal circle reading θ_p gives the magnetc bearing of line PQ.
  Therefore, the bearing of surveyline PQ = Horizontal circle reading at P
- Now shift the instrument to point Q. It must be noted that the Horizontal circle reading at this position is still θ_p. Now backsight to P.
- Transit the Theodolite. Now the line of sight of telescope is reversed (directed away from P). Now loosen the upperclamp screw and swing the telescope rightwards to sight point R and bisect it exactly. By doing this, the telescope has swept an additional angle θ_q. Hence the Horizontal circle reading now at Q is $\theta_p + \theta_q$.
  Therefore, the bearing of surveyline QR = Horizontal circle reading at Q
- Similar procedure is repeated at the next station R. Finally the horizontal circle reading at R becomes $\theta_p + \theta_q + \theta_r$, which is the bearing of surveyline RS.

2) ***Direct method without transiting***



- Let there be points P, Q, R, S, etc. on the field along which the traverse is required to be performed.

- Set the Theodolite at P and perform all temporary adjustments. Also set the horizontal circle reading to zero. Using the lower clamp and tangent screws, orient the line of sight to Magnetic Meridian (or North).
- Loosen the upper clamp screw and swing the telescope rightwards to sight point Q and bisect it exactly. Horizontal circle reading θ_p gives the magnetc bearing of line PQ.
  Therefore, the bearing of surveyline PQ = Horizontal circle reading at P
- Now shift the instrument to point Q. It must be noted that the Horizontal circle reading at this position is still θ_p. Now backsight to P. Loosen the upperclamp screw and swing the telescope rightwards to sight point R and bisect it exactly. By doing this, the telescope has swept an additional angle $\alpha + \theta_p + \beta$.
  Hence the Horizontal circle reading now at Q $= \theta_p + (\alpha + \theta_p + \beta)$.
  But, from Figure, we know, $\alpha + \theta_p = 180^o$

  Therefore, the Horizontal circle reading now at Q $= \theta_p + 180^o + \beta$
  $= (\theta_p + \beta) + 180^o$
  Also we know that the bearing of the line QR = $\theta_p + \beta$ (from figure)
  Therefore,
  Horizontal circle reading now at Q = Bearing of line QR + 180^o
  Hence it may be concluded that,
  Bearing of surveyline QR = Horizontal circle reading at Q – 180^o.
  Therefore, every even stations require a 180^ocorrection.
- Hence, it should be noted that,
  - at every odd stations (1^{st}, 3^{rd}, 5^{th},…), bearing of surveyline = Horizontal circle reading at that station
  - at every even stations (2^{nd}, 4^{th}, 6^{th},…), bearing of surveyline = Horizontal circle reading at that station ± 180^o
    - correction is +, if reading < 180^o.
    - correction is --, if reading > 180^o.

3) ***Backbearing Method***

- Let there be points P, Q, R, S, etc. on the field along which the traverse is required to be performed.
- Set the Theodolite at P and perform all temporary adjustments. Also set the horizontal circle reading to zero. Using the lower clamp and tangent screws, orient the line of sight to Magnetic Meridian (or North).
- Loosen the upper clamp screw and swing the telescope rightwards to sight point Q and bisect it exactly. Horizontal circle reading θ_p gives the magnetc bearing of line PQ.
  Therefore, the bearing of surveyline PQ = Horizontal circle reading at P = ForeBearing of line PQ
- Now shift the instrument to point Q. Now set vernier A reading to backbearing of line PQ, using
  Backbearing of a line = ForeBearing of line ± 180^o

- Now using lower clamp & tangent screws, orient the line of sight to point P. {At this position, the instrument is truly oriented to the North. Hence, any reading measured at this station will be the magnetic bearing}
- Now, loosen the upper clamp screw and sight point R and bisect it accurately. The horizontal circle reading gives the bearing of line QR.

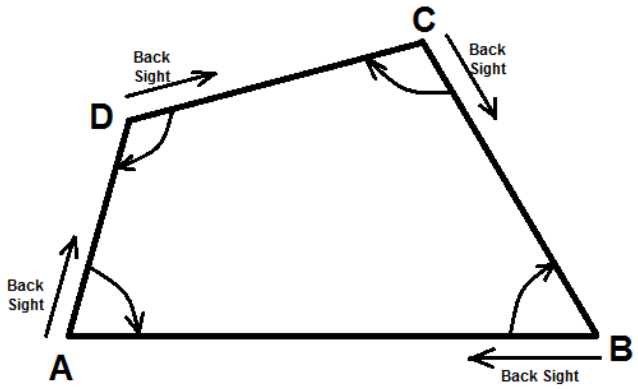
Direct observation of angles between lines

- This is the most accurate method
- Angles between lines are measured directly

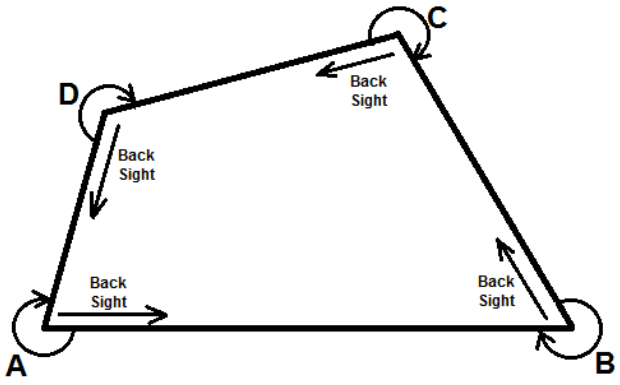
Angles are measured by any of the following methods.

1) *Traverse by Included angles*

- Measure angles directly from a backsight on the preceding station
- It is better to measure all angles clockwise, because graduations of horizontal circle increase in this direction
- Such angles measured clockwise may be interior or exterior angles, depending on the direction of progress of traverse (see figure)



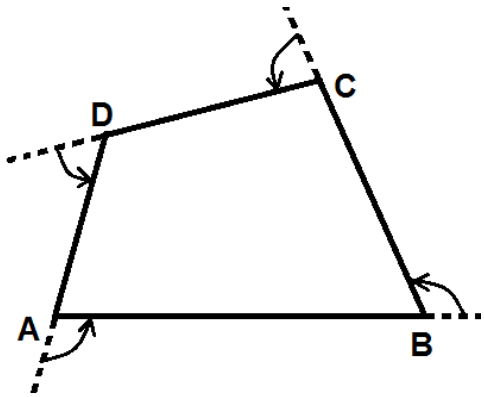
Direction of Traverse- ABCD (anticlockwise)
Direction of Angle Measurement - Clockwise
>> Angles are Interior



Direction of Traverse- ADCB (clockwise)
Direction of Angle Measurement - Clockwise
>> Angles are Exterior

2) *Traverse by Deflection angles*

- Angles which a survey line makes with the prolongation of the preceding line is measured.
- It is designated as left(L) or right(R), depending on whether deflection angle is measured anticlockwise or clockwise
- More suitable for survey of roads, railways, canals, pipelines etc.



TRAVERSE COMPUTATIONS

Any survey line on the field has a definite length (ℓ) and orientation (expressed in terms of bearing, that means the angle **θ** measured in the clockwise direction with respect to the North). Such a line has a projection parallel to the direction of the meridian (i.e., North-South direction), and another perpendicular to the direction of the meridian (i.e., East-West direction).

- Length of projection parallel to the direction of the meridian is called **LATITUDE (L)**
- Length of projection perpendicular to the direction of the meridian is called **DEPARTURE (D)**

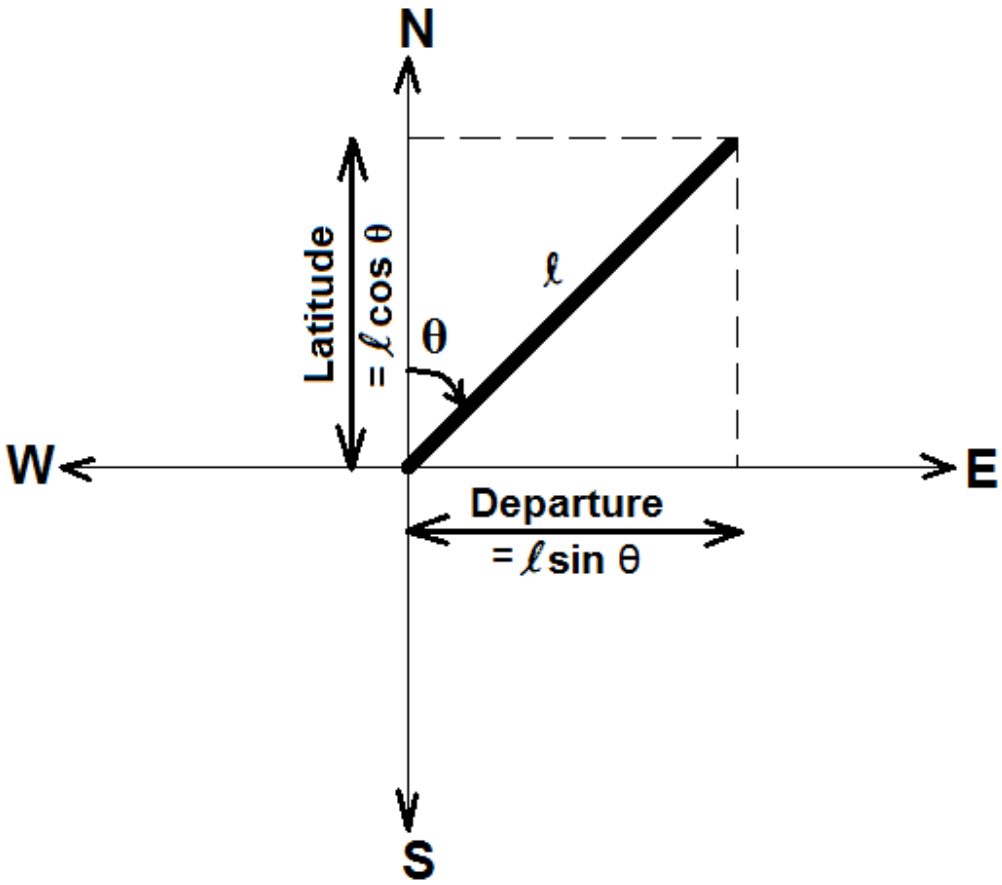
From figure, it is clear that

$L = \ell \cos \theta$

$D = \ell \sin \theta$

It may be noted that

- L is + when measured Northward
- L is – when measured Southward
- D is + when measured Eastward
- D is – when measured Southward



- L measured Northward is called *Northing*; L measured Southward is called *Southing*
- D measured Eastward is called *Easting*; D measured Westward is called *Westing*

Depending on the value of angle **θ**, the sign of Latitude and Departure is as follows:

Whole Circle Bearing W.C.B. (angle **θ**)	Reduced Bearing R.B.	Latitude (L)	Departure (D)
$0 – 90^o$	N θ E	+	+
$90^o – 180^o$	S θ E	--	+
$180^o – 270^o$	S θ W	--	--
$270^o – 360^o$	N θ W	+	--

Coordinates of Traverse Stations

The relative positions of traverse station points on the field may be represented in terms of a coordinate system, which involves 2 indices – Latitude coordinate & Departure coordinate. Thus any station point on the field needs these two coordinate values for representation.

a. Consecutive Coordinates (or Dependent Coordinates)

- Here, for every station point, the origin is taken as the preceding station point on the survey line joining these two points.
- Latitude coordinate of any station point = Latitude of the survey line joining this point and preceding point.
- Departure coordinate of any station point = Departure of the survey line joining this point and preceding point.

b. Total Coordinates (or Independent Coordinates)

- Here, there is a common origin for all station points. Any one of the station points is generally chosen as the origin.
- Latitude coordinate of any station point = Algebraic sum of Latitudes of all survey lines joining this point and origin.
- Departure coordinate of any station point = Algebraic sum of Departures of all survey lines joining this point and origin.

Example: Determine dependent & independent coordinates of traverse stations of the following traverse.

Line	AB	BC	CD	DA
Length (m)	232	148	417	306.70
WCB	$32^O12'$	$138^O36'$	$202^O24'$	$348^O13'$

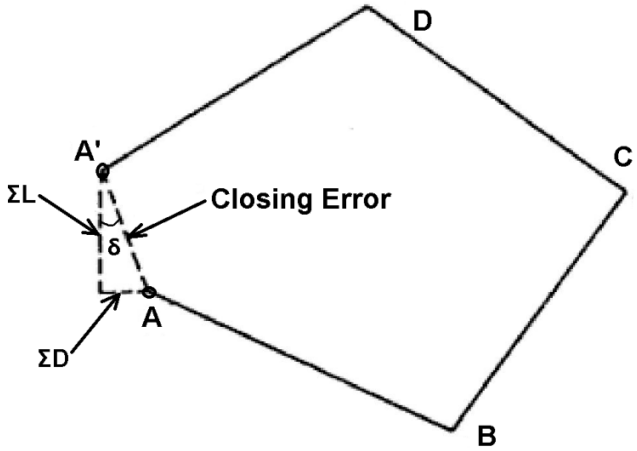
Ans:

Line	Length, ℓ (m)	WCB (θ)	Latitude (=ℓ cos θ)	Departure (=ℓ sin θ)	Traverse Station	Consecutive Coordinates		Total Coordinates	
						Latitude Coord	Departure Coord	Latitude Coord	Departure Coord
					A			400 (assumed)	400 (assumed)
AB	232	$32^{O}12'$	196.32	123.63					
					B	196.32	123.63	596.32	523.63
BC	148	$138^{O}36'$	-111.02	97.87					
					C	-111.02	97.87	485.3	621.5
CD	417	$202^{O}24'$	-385.54	-158.91					
					D	-385.54	-158.91	99.76	462.59
DA	306.7	$348^{O}13'$	300.24	-62.59					
					A	300.24	-62.59	400	400

ERROR OF CLOSURE

If a **closed traverse** is plotted according to the field measurements, the end point of the traverse may not coincide with the first point, owing to errors in observations of angles & distances. Such error is called **Error of Closure** or **Closing Error**.

- This is determined by computing the algebraic sum of latitudes of all surveylines involved in the traverse (ΣL) and the algebraic sum of departures of all surveylines (ΣD).
- Physically, ΣL represents the component of closing error parallel to the direction of the meridian; ΣD represents the component of closing error perpendicular to the direction of the meridian (see figure).
- Closing Error e = $\sqrt{(\Sigma L)^2 + (\Sigma D)^2}$
- Direction of closing error is given by: tan δ = $\frac{(\Sigma D)}{(\Sigma L)}$
- Relative Error of Closure = $\frac{\text{Error of Closure}}{\text{Perimeter of Traverse}} = \frac{e}{p}$,where p = AB + BC + CD + DA'
- For a perfectly closed traverse (where the first & last points of traverse coincide exactly),
  - Algebraic sum of latitudes of all surveylines ΣL= 0
  - Algebraic sum of departures of all surveylines ΣD = 0



ADJUSTMENTS OF ERRORS

This involves adjustments of included angles/bearings and applying corrections to Latitudes & Departures for closed traverse.

a) Adjustment of Angles or Bearings

a) Adjustments of Angles

- Performed for traverses plotted using method of direct observation of included angles.
- Should be performed before calculating Latitudes & Departures of surveylines.
- This is performed as follows:
  - Compute the sum of all interior included angles obtained from observations. (actual sum)
  - Also compute the theoretical sum of all interior included angles by formulae, (required sum)

  Sum of all interior included angles of a closed polygon = $(2N-4) \times 90^o$

  $= (N-2) \times 180^o$

  Where N is the total number of sides of the polygon.

  - Now, compare the two. Angular error = Actual sum –Required sum
  - Correction = – Angular Error
  - If angles are measured with same degree of precision, the correction is equally distributed among all angles of traverse.

b) Adjustments of Bearings

- Performed if Fast Needle method is used for traverse (where bearings are measured instead of included angles)
- Closing error in bearing (e_b) is computed as = Difference between bearings of the last line as observed from first and last stations
- Correction is applied to bearings as follows:
  - Correction for 1st line = $\frac{e_b}{N}$
  - Correction for 2nd line = $\frac{2e_b}{N}$
  - Correction for 3rd line = $\frac{3e_b}{N}$
  - Correction for last line = $\frac{Ne_b}{N} = e_b$

b) Balancing Traverse (applying corrections to Latitudes & Departures to make ΣL= 0 & ΣD= 0). Any of the following methods may be chosen.

c) Bowditch's Method (or Compass Rule)

- Applied if linear measurements are of same precision as angular measurements.
- Here, total error in Latitude (or departure) of any survey line is distributed in proportion of lengths of the lines. Therefore,

Correction to Latitude of any side = Total error in Latitude x $\frac{Length\ of\ that\ side}{\text{Perimeter of Traverse}}$

Correction to Departure of any side = Total error in Departure x $\frac{Length\ of\ that\ side}{\text{Perimeter of Traverse}}$

d) Transit Method

- Applied if angular measurements are more precise than linear measurements.
- Here, total error in Latitude (or departure) of any survey line is distributed in proportion of Latitudes (or Departure) of the lines. Therefore,

Correction to Latitude of any side = Total error in Latitude x $\frac{Latitude\ of\ that\ side}{Arithmetic\ sum\ of\ latitudes\ of\ all\ lines}$

Correction to Departure of any side=Total error in Departure x $\frac{Departure\ of\ that\ side}{Arithmetic\ sum\ of\ departures\ of\ all\ lines}$

GALES TRAVERSE TABLE

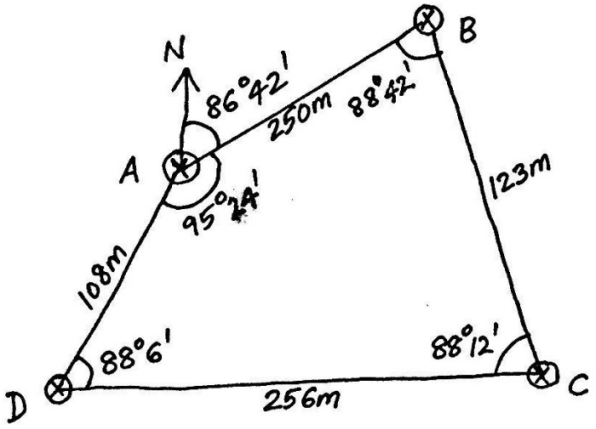
Traverse computations are performed in a tabular form, known as *Gales Traverse Table*.

It comprises the following steps:

a) List all the survey lines of traverse and its lengths
b) List all traverse stations and included angles measured at each of them
c) Adjust the included angles to satisfy the geometrical conditions
d) With the bearing of one traverse line known, determine the bearings of all other survey lines
e) Compute Latitude & Departure of all survey lines, and hence write down the consecutive coordinates of all traverse stations
f) Calculate ΣL and ΣD. Now balance the traverse using Bowditch's method or Transit method.
g) From this corrected coordinates, determine independent coordinates of all traverse station points.

See the following example.

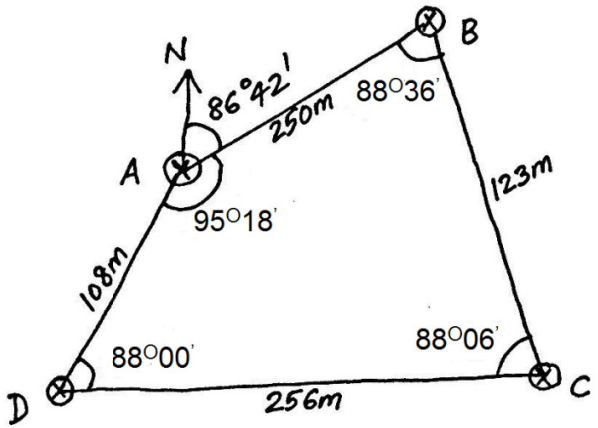
Q: Prepare Gale's Traverse Table for the following traverse.



Ans:

Prepare Gales Traverse Table as shown below.

Corrected Included angles are as shown:



Based on these corrected included angles, Bearings of lines are computed.

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Line (GIVEN)	Length (m) (GIVEN)	Point	Included Angle (GIVEN)	Correction	Corrected Included Angle	WCB	Latitude (N)	Departure (E)	Correction to Latitude (N)	Correction to Departure (E)	Corrected Consecutive Coordinates N	Corrected Consecutive Coordinates E	Independent Coordinates N	Independent Coordinates E
~~AB~~		A	95°24′	−6′	95°18′						108.198	3.759	0 (ASSUMED)	0 (ASSUMED)
AB	250					86°42′ (GIVEN)	14.39	249.59	+0.036	−0.700				
		B	88°42′	−6′	88°36′						14.426	248.890	14.426	248.890
BC	123					178°6′	−122.93	4.08	+0.306	−0.011				
		C	88°12′	−6′	88°6′						−122.624	4.069	−108.198	252.959
CD	256					270°00′	0	−256.00	0	−0.718				
		D	88°6′	−6′	88°0′						0	−256.718	−108.198	−3.759
DA	108					2°00′	107.93	3.77	+0.268	−0.011				
			Σ = 360°24′			Algebraic Σ (Error) =	−0.61	1.44						
						Arithmetic Σ =	245.25	513.44						

Required sum
= (4−2)×180°
= 360°

Error = 360°24′ − 360°
= 24′

Correction = −24′ to be distributed equally to 4 angles.

∴ Correction to each angle = $\frac{-24'}{4} = -6'$

Using Transit Rule,

$$\text{Correction to Lat (or Dep) of any Line} = -\text{Total Error} \times \frac{\text{Lat (or Dep) of the Line}}{\text{Arithmetic } \Sigma \text{ of Lat (or Dep)}}$$

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Note: If the independent coordinates of a closed traverse are known, then **area of traverse** is found out as follows.

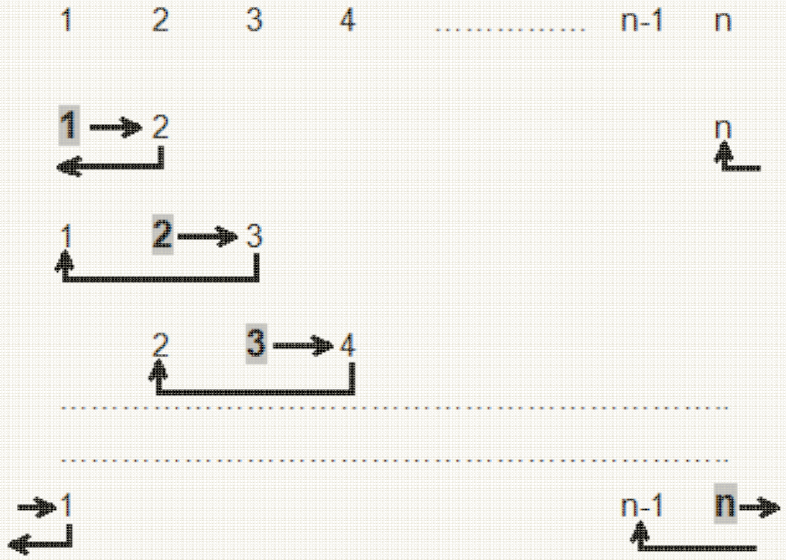
Let $x_1, x_2, x_3 \ldots x_n$ are the E-coordinates or East Coordinates of traverse stations.

Let $y_1, y_2, y_3 \ldots y_n$ are the N-coordinates or North Coordinates of traverse stations.

Then the area of the closed traverse is:

$$A = \frac{1}{2}[y_1(x_2 - x_n) + y_2(x_3 - x_1) + y_3(x_4 - x_2) + \ldots\ldots\ldots\ldots\ldots + y_n(x_1 - x_{n-1})]$$

To remember the above formula, memorize the following pattern:



Example:

In the above question, area of traverse is found out as follows.

$$A = \frac{1}{2}[y_1(x_2 - x_4) + y_2(x_3 - x_1) + y_3(x_4 - x_2) + y_4(x_1 - x_3)]$$

$$= \frac{1}{2}[0(248.89 - (-3.759)) + 14.426(252.959 - 0) + (-108.198)(-3.759 - 248.89) + (-108.198)(0 - 252.959)]$$

$$= \underline{29177.48\ m^2}$$

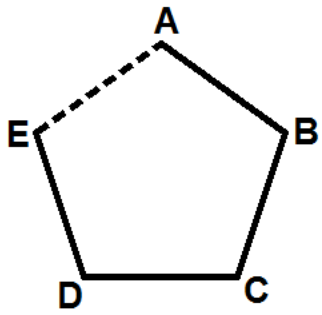
OMITTED MEASUREMENTS

Measurements, such as Length or Bearing, omitted in the process of survey may be computed in office work. But there are following limitations:

- Such missing quantities should not be more than two
- There is no way to check for errors and eliminate them, because these become part of computed values of missing quantities

Missing quantities may be computed as follows.

a) If Length of one side is omitted OR
If Bearing of one side is omitted OR
If Length & Bearing of one side is omitted



In the traverse shown, let the length/bearing/length & bearing of AE is missing. For a closed traverse,

$\Sigma L = 0$

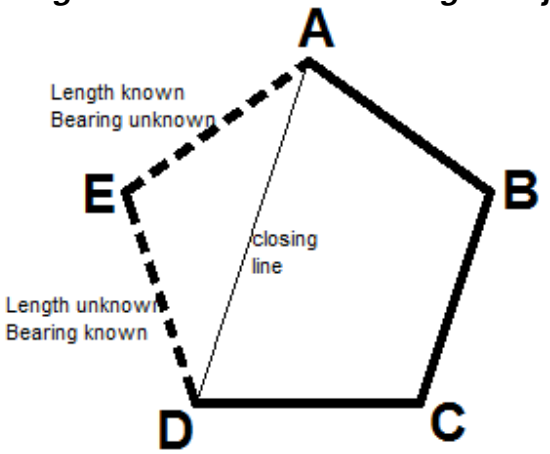
$\Rightarrow L_{AB} + L_{BC} + L_{CD} + L_{DE} + L_{EA} = 0$

These are the only unknowns. All other terms are known. Hence these can be computed easily from these equations

$\Sigma D = 0$

$\Rightarrow D_{AB} + D_{BC} + D_{CD} + D_{DE} + D_{EA} = 0$

b) If length of one side and bearing of adjacent side are omitted



In the traverse shown, length of DE and bearing of AE are unknown.

- Join AD such that all quantities of traverse ABCD are known, except AD. Length & Bearing of AD are computed as in previous case ***a)***.
- Since bearing of ED & AD are known, ∟EDA is known.
- Therefore, from sine rule

$$\frac{EA}{\sin \angle EDA} = \frac{AD}{\sin \angle AED}$$

⇨ From this, ∟AED can be found out. Hence bearing of line EA can be computed.

- Also, ∟EAD is found out from $\angle EAD = 180^{o} - \angle AED - \angle EDA$
- From sine rule,

$$\frac{DE}{\sin \angle EAD} = \frac{AD}{\sin \angle AED}$$

=> From this, side DE can be calculated

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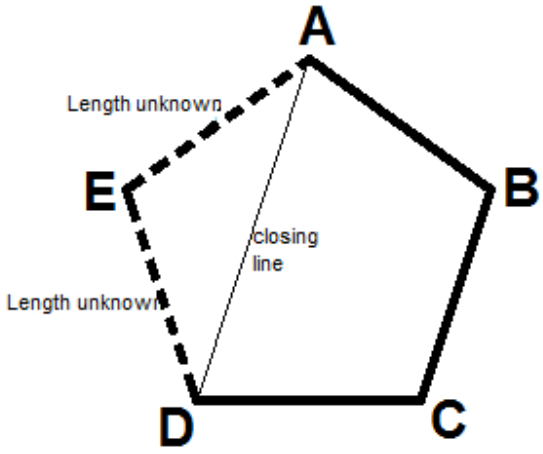
c) If length of two adjacent sides are omitted



In the traverse shown, length of DE and AE are unknown.

- Join AD such that all quantities of traverse ABCD are known, except AD. Length & Bearing of AD are computed as in previous case ***a)***.
- Since bearing of AD, ED & AE are known, ∟AED, ∟EDA & ∟EAD are known.
- Therefore, unknown lengths can be obtained from from sine rule

$$\frac{AE}{\sin \angle \mathrm{EDA}} = \frac{AD}{\sin \angle \mathrm{AED}}$$

$$\frac{ED}{\sin \angle \mathrm{EAD}} = \frac{AD}{\sin \angle \mathrm{AED}}$$

CHAPTER 7

TRIGONOMETRIC LEVELLING

Trigonometric leveling – the process of determining the relative elevations of station points, from observed vertical angles and horizontal distances.

- Vertical angles are measured using Theodolite
- Horizontal distances are measured using Tape/Chain.

This may categorized under *Plane Surveying* or *Geodetic Surveying*. However, ***Trigonometric leveling under Plane Surveying*** is only dealt in this chapter, since we consider that the observed distances between points are considerably small such that the effects of curvature and refraction may either be neglected or corrections applied.

HEIGHTS AND DISTANCES

Trigonometric leveling is used to determine the difference in elevation between instrument axis and top of the given object. This can be studied from the following cases:

1) **Base of object is accessible** *(Here, the horizontal distance from the instrument station to the base of the object can be measured)*
2) **Base of object is not accessible** *(Here, the horizontal distance from the instrument station to the base of the object cannot be measured)*
    a. Instrument stations in the same vertical plane as the object
    b. Instrument stations are not in same vertical plane as the object

Note: If the measured horizontal distance is large, the *combined correction for curvature & refraction* must be applied to the elevations computed with respect to the instrument axis. The magnitude of correction is:

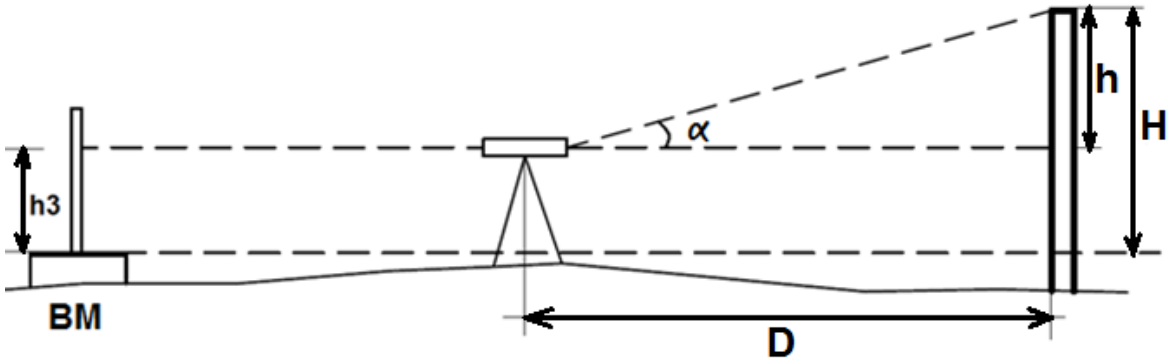
$$C = 0.0673\ D^2$$, where D is the horizontal distance in km

- Correction is +, if the vertical angle measured is angle of elevation
- Correction is -, if the vertical angle measured is angle of depression

1) Base of the object is accessible

- Here, the horizontal distance from the instrument station to the base of the object can be measured accurately.
- Let,
    H = Height of the object above Benchmark (BM)
    h = Height of the object above the instrument axis
    h_3 = Staff reading on the BM
    α = Vertical angle observed at instrument station to the top of given the object
    D = horizontal distance measured from the instrument station to the base of the given object

- See the figure. The instrument is set at definite distance away from the given object. The vertical angle to the top of the object is measured. Also, the horizontal distance from the instrument station to the base of the object is measured.



From figure,

$$\tan \alpha = \frac{h}{D} \quad \Rightarrow \mathbf{h = D \tan \alpha}$$

Therefore, $H = h + h_3$

RL of top of object = H + RL of BM

Note: *If D is very large, combined correction due for curvature and refraction C should be applied. Therefore,* **$h = D \tan \alpha + C$**

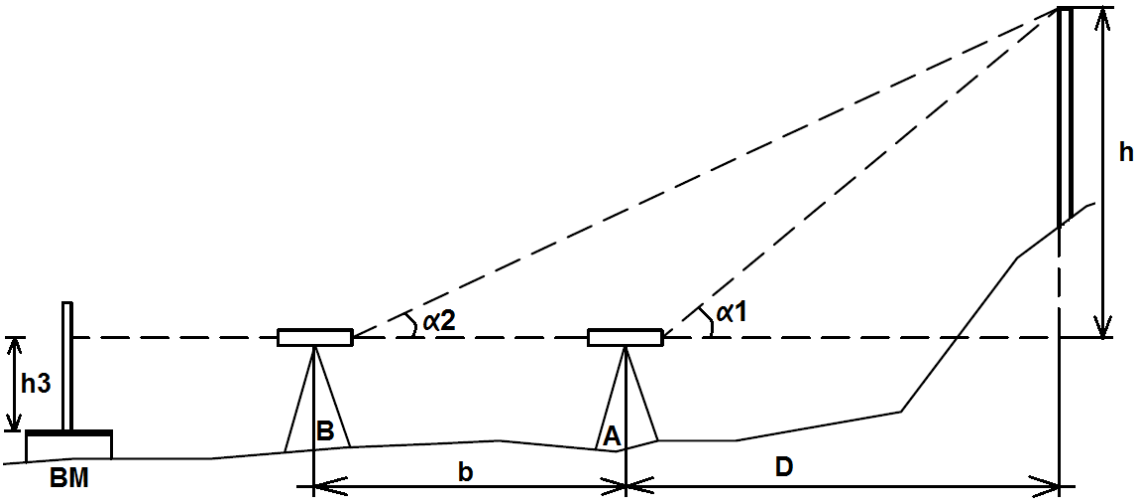
2) Base of the object is not accessible

- Here, the horizontal distance from the instrument station to the base of the object cannot be measured. Hence, observations at more than one instrument station are required here.

a) Instrument stations in the same vertical plane as the object:

⊙ ***Instrument Axis at the same level:***

- Here, the Theodolite is set exactly at the same level at two stations



- The instrument is set at definite distance away from the given object. The vertical angle to the top of the object is measured at A. Theodolite is transited and new instrument station B is established. The horizontal distance between A and B is measured. Set the instrument at B and observe vertical angle to the object.

h = D tan $\alpha 1$ => $D = \frac{h}{\tan \alpha 1}$ ----------(1)

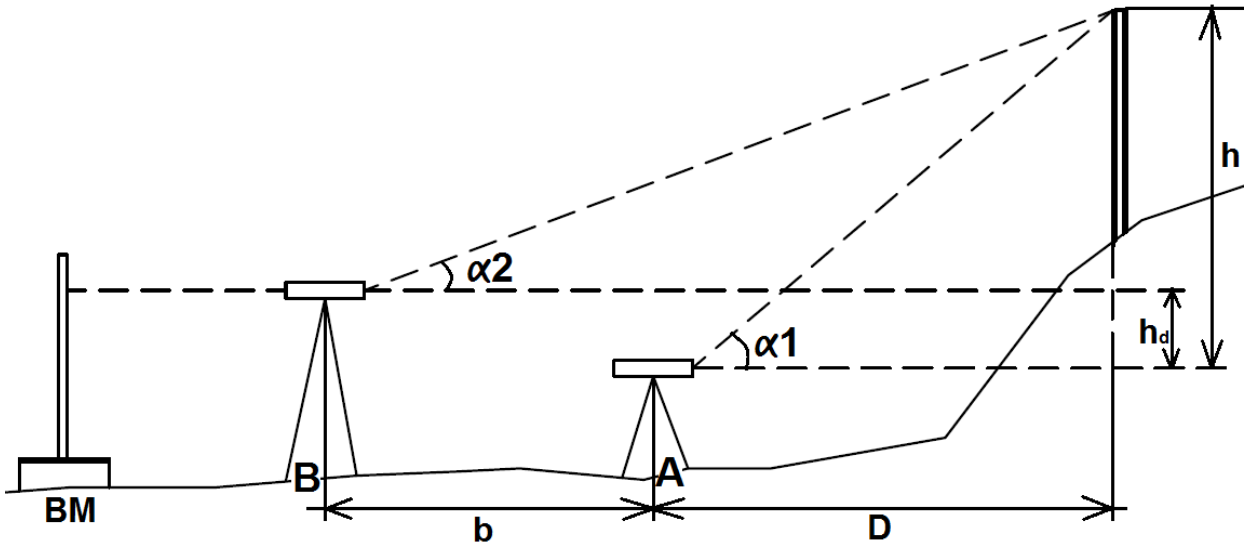
h = (D + b) tan $\alpha 2$ ----------(2)

Substituting (1) in (2) => $h = [\frac{h}{\tan \alpha 1} + b] \tan \alpha 2$

$$\Rightarrow \quad \mathbf{h = \frac{b \tan \alpha 1 \tan \alpha 2}{\tan \alpha 1 - \tan \alpha 2}}$$

⊙ ***Instrument Axis at the different levels:***

- Since the process of setting the theodolites exactly at the same level at two stations is very difficult and time-consuming, this method is resorted to.



h = D tan $\alpha 1$ => $D = \frac{h}{\tan \alpha 1}$ ----------(1)

$h - h_d$ = (D + b) tan $\alpha 2$ ----------(2)

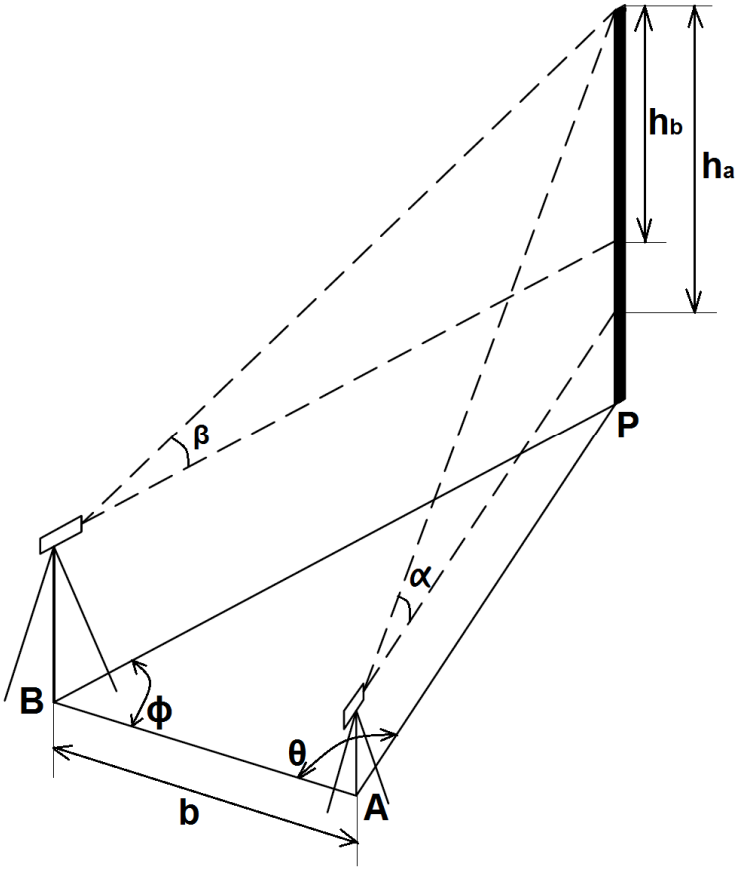
Substituting (1) in (2) => $h - h_d = [\frac{h}{\tan \alpha 1} + b] \tan \alpha 2$

$$\Rightarrow \quad \mathbf{h = \frac{(b \tan \alpha 2 \pm h_d) \tan \alpha 1}{\tan \alpha 1 - \tan \alpha 2}}$$

In the above formulation, + is used if the farther instrument station is higher; - is used if the farther instrument station is lower.

b) <u>Instrument stations are not in the same vertical plane as the object:</u>

- Let,
  - h_a = Height of the object above the instrument axis of A
  - h_b = Height of the object above the instrument axis of B
  - α = Vertical angle observed at instrument station A to the top of the object
  - β = Vertical angle observed at instrument station B to the top of the object
  - θ = Horizontal angle observed at instrument station A between B & the object
  - ϕ = Horizontal angle observed at instrument station B between A & the object
  - b = horizontal distance measured between instrument stations A & B
- See the figure. The instrument is set at definite distance away from the given object at two stations. The vertical angle to the top of the object is measured. Also, the horizontal distance between instrument stations.



From sine rule, $\frac{b}{\sin[\pi-(\theta+\phi)]} = \frac{BP}{\sin\theta} = \frac{AP}{\sin\phi}$

=> $AP = \frac{b\ \sin\phi}{\sin[\pi-(\theta+\phi)]}$ and $BP = \frac{b\ \sin\theta}{\sin[\pi-(\theta+\phi)]}$

We know, $h_a = AP \tan\alpha$ => $\mathbf{h_a = \frac{b\ \sin\phi\ \tan\alpha}{\sin[\pi-(\theta+\phi)]}}$

and $h_b = BP \tan\beta$ => $\mathbf{h_b = \frac{b\ \sin\theta\ \tan\beta}{\sin[\pi-(\theta+\phi)]}}$

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Example 5.1. *An instrument was set up at P and the angle of elevation to a vane 4 m above the foot of the staff held at Q was 9° 30′. The horizontal distance between P and Q was known to be 2000 metres. Determine the R.L. of the staff station Q, given that the R.L. of the instrument axis was 2650.38.*

Solution.

Height of vane above the instrument axis

$$= D \tan \alpha = 2000 \tan 9° 30' = 334.68 \text{ m}$$

Correction for curvature and refraction $= \frac{6}{7} \frac{D^2}{2R}$

or $C = 0.06728\ D^2$ m, when D is in km

$$= 0.06728 \left(\frac{2000}{1000} \right)^2 = 0.269 \approx 0.27 \text{ m } (+ \text{ve})$$

Height of vane above the instrument axis = 334.68 + 0.27 = 334.95

R.L. fo vane = 334.95 + 2650.38 = 2985.33 m

R.L. of Q = 2985.33 − 4 = **2981.33 m.**

Example 5.2. *An instrument was set up at P and the angle of depression to a vane 2 m above the foot of the staff held at Q was 5° 36′. The horizontal distance between P and Q was known to be 3000 metres. Determine the R.L. of the staff station Q, given that staff reading on a B.M. of elevation 436.050 was 2.865 metres.*

Solution.

The difference in elevation between the vane and the instrument axis

$$= D \tan \alpha = 3000 \tan 5° 36' = 294.153$$

Combined correction due to curvature and refraction $= \frac{6}{7} \frac{D^2}{2R}$

or $C = 0.06728\ D^2$ metres, when D is in km $= 0.06728 \left(\frac{3000}{1000} \right)^2 = 0.606$ m.

Since the observed angle is negative, the combined correction due to curvature and refraction is subtractive.

Difference in elevation between the vane and the instrument axis = 294.153 − 0.606

= 293.547 = h.

R.L. of instrument axis = 436.050 + 2.865 = 438.915

∴ R.L. of the vane = R.L. of instrument aixs. − h

= 438.915 − 293.547 = 145.368

R.L. of Q = 145.368 − 2

= **143.368 m.**

Example 5.3. *In order to ascertain the elevation of the top (Q) of the signal on a hill, observations were made from two instrument stations P and R at a horizontal distance 100 metres apart, the station P and R being in the line with Q. The angles of elevation of Q at P and R were 28 ° 42′ and 18 ° 6′ respectively. The staff reading upon the bench mark of elevation 287.28 were respectively 2.870 and 3.750 when the instrument was at P and at R, the telescope being horizontal. Determine the elevation of the foot of the signal if the height of the signal above its base is 3 metres.*

Solution. (Fig. 5.6)

Elevation of instrument axis at P = R.L of B.M. + Staff reading

$= 287.28 + 2.870 = 290.15$ m

Elevation of instrument axis at R = R.L. of B.M. + staff reading

$= 287.28 + 3.750 = 291.03$ m

Difference in level of the instrument axes at the two stations $= s = 291.03 - 290.15 = 0.88$ m

$\alpha_1 = 28° 42'$ and $\alpha_2 = 18° 6'$

$s \cot \alpha_2 = 0.88 \cot 18° 6' = 2.69$ m

From equation 5.4 (*a*), we have

$$D = \frac{(b + s \cot \alpha_2) \tan \alpha_2}{\tan \alpha_1 - \tan \alpha_2} = \frac{(100 + 2.69) \tan 18° 6'}{\tan 28° 42' - \tan 18° 6'} = \mathbf{152.1} \text{ m.}$$

$\therefore \quad h_1 = D \tan \alpha_1 = 152.1 \tan 28° 42' = 83.272$ m

$\therefore$ R.L. of foot of signal = R.L. of inst. axis at $P + h_1$ – ht. of signal

$= 290.15 + 83.272 - 3 = \mathbf{370.422}$ m.

Check : $(b + D) = 100 + 152.1 = 252.1$ m

$h_2 = (b + D) \tan \alpha_2 = 252.1 \times \tan 18° 6' = 82.399$ m

R.L. of foot of signal = R.L. of inst. axis at $R + h_2$ + ht. of signal

$= 291.03 + 82.399 - 3 = \mathbf{370.429}$ m.

Example 5.4. *The top (Q) of a chimney was sighted from two stations P and R at very different levels, the stations P and R being in the line with the top of the chimney. The angle of elevation from P to the top of the chimney was 38 ° 21′ and that from R to the top of the chimney was 21 ° 18′ . The angle of elevation from R to a vane 2 m above the foot of the staff held at P was 15 ° 11′. The heights of the instrument at P and R were 1.87 m and 1.64 m respectively. The horizontal distance between P and R was 127 m and the reduced level of R was 112.78 m. Find the R.L. of the top of the chimney and the horizontal distance from P to the chimney.*

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Solution. (Figs. 5.8 and 5.9)

(*i*) When the observations were taken from R to P

$$h = b \tan \alpha = 127 \tan 15° 11' = 34.47 \text{ m}$$

R.L. of P = R.L. of R + height of instrument at $R + h - r$

$$= 112.78 + 1.64 + 34.47 - 2 = 146.89 \text{ m}$$

R.L. of instrument axis at P = R.L. of P + ht. of instrument at P

$$= 146.89 + 1.87 = 148.76 \text{ m} \qquad ...(i)$$

Difference in elevation between the instrument axes $= s$

$$= 148.76 - (112.78 + 1.64) = 34.34 \text{ m}$$

$$\therefore \quad D = \frac{(b \tan \alpha_2 - s)}{\tan \alpha_1 - \tan \alpha_2} = \frac{127 \tan 21° 18' - 34.34}{\tan 38° 21' - \tan 21° 18'} = \frac{49.25 - 34.34}{0.79117 - 0.38988} = 37.8 \text{ m}$$

$$h_1 = D \tan \alpha_1 = 37.8 \tan 38° 21' = 29.91$$

$\therefore$ R.L. of Q = R.L. of instrument axis at $P + h_1$

$$= 148.76 + 29.91 = \mathbf{178.67} \text{ m}.$$

Check : R.L. of Q = R.L. of instruments axis at $R + h_2$

$$= (112.78 + 1.64) + (b + D) \tan \alpha_2 = 114.42 + (127 + 37.8) \tan 21° 18'$$

$$= 114.42 + 64.25 = \mathbf{178.67} \text{ m}.$$

Example 5.5. *To find the elevation of the top (Q) of a hill, a flag staff of 2 m height was erected and observations were made from two stations P and R, 60 metres apart. The horizontal angle measured at P between R and the top of the flat staff was 60° 30′ and that measured at R between the top of the flag staff and P was 68° 18′. The angle of elevation to the top of the flag staff was measured to be 10° 12′ at P. The angle of elevation to the top of the flag staff was measured to be 10° 48′ at R. Staff readings on B.M. when the instrument was at P = 1.965 m and that with the instrument at R = 2.055 m. Calculate the elevation of the top of the hill if that of B.M. was 435.065 m.*

Solution. (Fig. 5.10)

Given : $b = 60$ m

$\theta_1 = 60° 30'$; $\theta_2 = 68° 18'$

$\alpha_1 = 10° 12'$; $\alpha_2 = 10° 18'$

$$\therefore \quad PQ_1 = D = \frac{b \sin \theta_2}{\sin (\theta_1 + \theta_2)}$$

and $$h_1 = D \tan \alpha_1 = \frac{b \sin \theta_2 \tan \alpha_1}{\sin (\theta_1 + \theta_2)} = \frac{60 \sin 68° 18' \tan 10° 12'}{\sin (60° 30' + 68° 18')} = 12.87 \text{ m}$$

$\therefore$ R.L. of Q = (R.L. of instrument axis at P) + h_1

$$= (435.065 + 1.965) + 12.87 = \mathbf{449.900} \text{ m}.$$

Check

$$h_2 = \frac{b \sin \theta_1 \tan \alpha_2}{\sin (\theta_1 + \theta_2)} = \frac{60 \sin 60° 30' \tan 10° 48'}{\sin (60° 30' + 68° 18')} = 12.78 \text{ m}$$

$\therefore$ R.L. of Q = R.L. of instrument axis at $R + h_2 = (435.065 + 2.055) + 12.78$

$$= \mathbf{449.9} \text{ m}.$$

CHAPTER 8

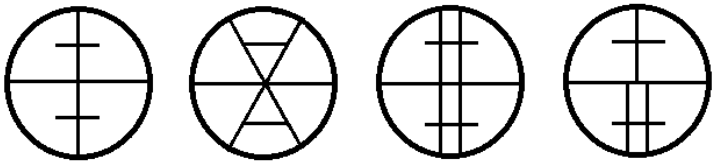
TACHEOMETRY

Tacheometry is the branch of survey in which the horizontal and vertical distances of points are obtained by instrumental observations such as staff intercepts and angles.

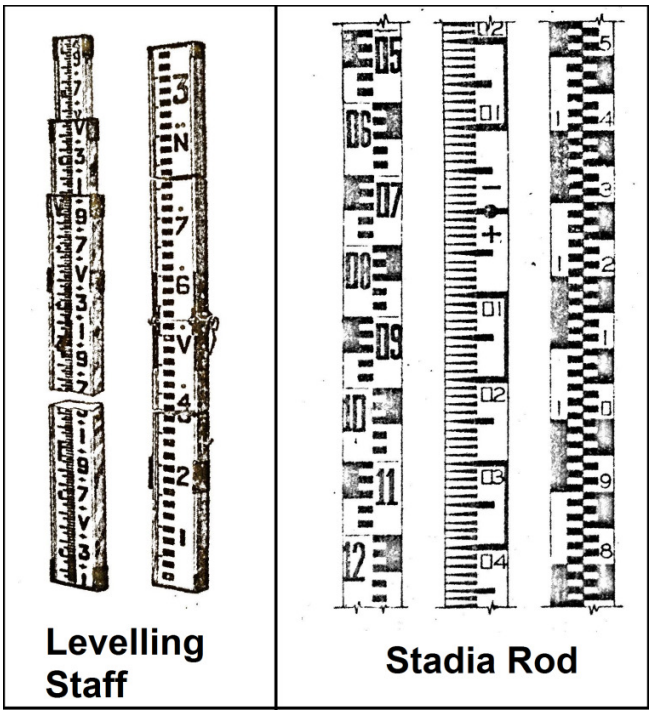
- The process of measuring horizontal distance is eliminated
- Less accurate, but more rapid
- Best useful in case of rough & difficult terrain
- Primary objective is preparation of contour maps or plans
- Used in Hydrographic surveys, location surveys etc.

Instruments used in Tacheometry are:

- *Tacheometer* – a transit Theodolite fitted with a stadia diaphragm
  - ***Diaphragm*** is the frame upon which is mounted the crosshairs to give a definite line of sight.
  - ***Stadia hairs / Stadia Lines*** are a pair of horizontal hairs provided in addition to the regular crosshairs. Each on of this is equidistant from the central horizontal hair. Commonly used Stadia diaphragms are shown here.



- *Levelling Staff / Stadia Rod* – a scale, shown below, graduated in metres and centimeters. They may be single solid, folding or telescopic.



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Important Terminology

- **Stadia Interval (i)**– the actual physical distance between the top & bottom stadia hairs
- **Staff intercept (S)** – the difference between the leveling staff readings corresponding to the top & bottom stadia hairs

CLASSIFICATION OF TACHEOMETRY – based on the underlying principle

1) **Stadia Tacheometry** – make use of stadia hairs. Involves observation of either staff intercepts or stadia intervals.
   a. *Fixed Hair method* –
      - Stadia hairs are stationary => Stadia interval (i) is constant
      - Staff intercept depends upon the horizontal distance between the tacheometer and stadia rod => Staff intercept (S) is variable
      - Most commonly used
   b. *Movable Hair method* –
      - Here, the stadia rod or leveling staff is provided with two fixed targets. The stadia hairs can be moved to coincide the targets accurately, using micrometer screws => Stadia interval (i) is variable
      - Since the same staff with fixed targets is used everywhere => Staff intercept (S) is constant
      - Rarely used
2) **Tangential Tacheometry** – does not use stadia hairs, hence stadia diaphragm is not required. Here, a transit Theodolite is used to measure vertical angles to the two fixed targets on a staff held at a station point.

STADIA TACHEOMETRY

Principle: The principle of Stadia method of Tacheometry is as follows:

Let i = Stadia interval

S = Staff intercept

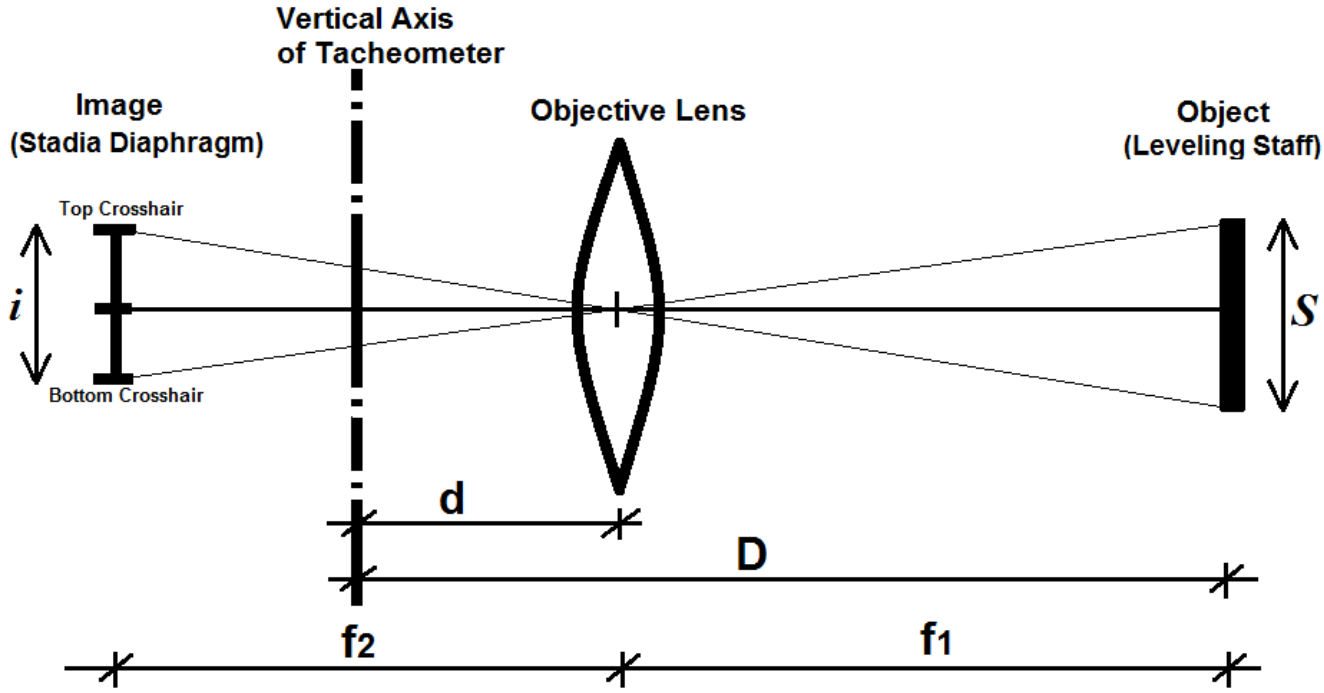
f = Focal length of the objective lens

f_1 = horizontal distance between objective lens and object (leveling staff)

f_2 = horizontal distance between objective lens and image (stadia diaphragm)

d = horizontal distance between objective lens and vertical axis of tacheometer

D = horizontal distance between vertical axis of tacheometer and object (leveling staff)



From similar triangles in the above figure,

$\frac{i}{f_2} = \frac{S}{f_1}$ ----------(1)

From lens formula,

$\frac{1}{f} = \frac{1}{f_1} + \frac{1}{f_2}$ ----------(2)

Solving both equations (1) and (2), we get

$f_1 = (\frac{f}{i})\, S + f$ ----------(3)

The horizontal distance between the vertical axis of the Tacheometer and the staff (object) is

$D = f_1 + d$

Substituting equation (3) $\Rightarrow D = (\frac{f}{i})\, S + (f + d)$

$\Rightarrow$ **D = K S + C**

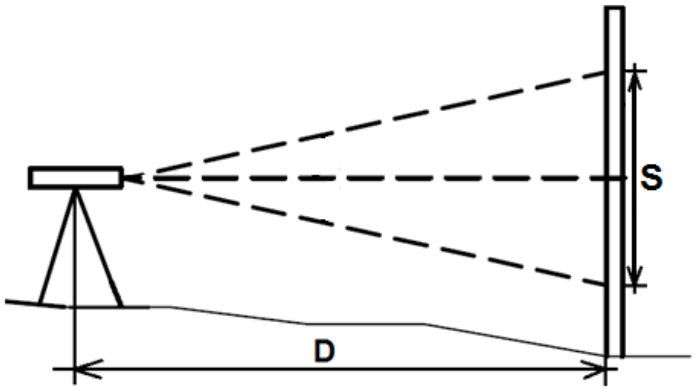
where, K = $\frac{f}{i}$ = Multiplying Constant

and, C = (f + d) = Additive Constant

- Generally, the value of K = 100 and C = 0.
- Anallatic lens is a convex lens provided between objective & eyepiece of a tacheometer, in order to eliminate the additive constant (i.e., to make the value of C to be zero).
- The above equation concludes that, the horizontal distance in tacheometry is a function of Staff intercept only.

DIFFERENT CASES INVOLVED:

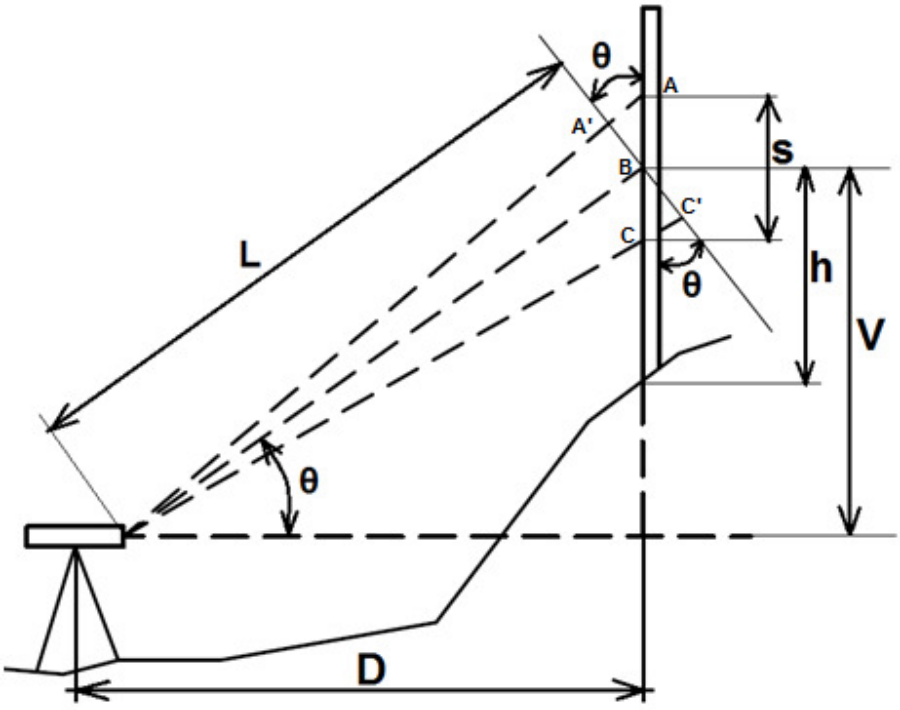
1) *Line of Sight is Horizontal*



⊙ **D = K S + C**

2) *Line of Sight is Inclined*

a) Staff held vertical:



AC = S = Staff intercept

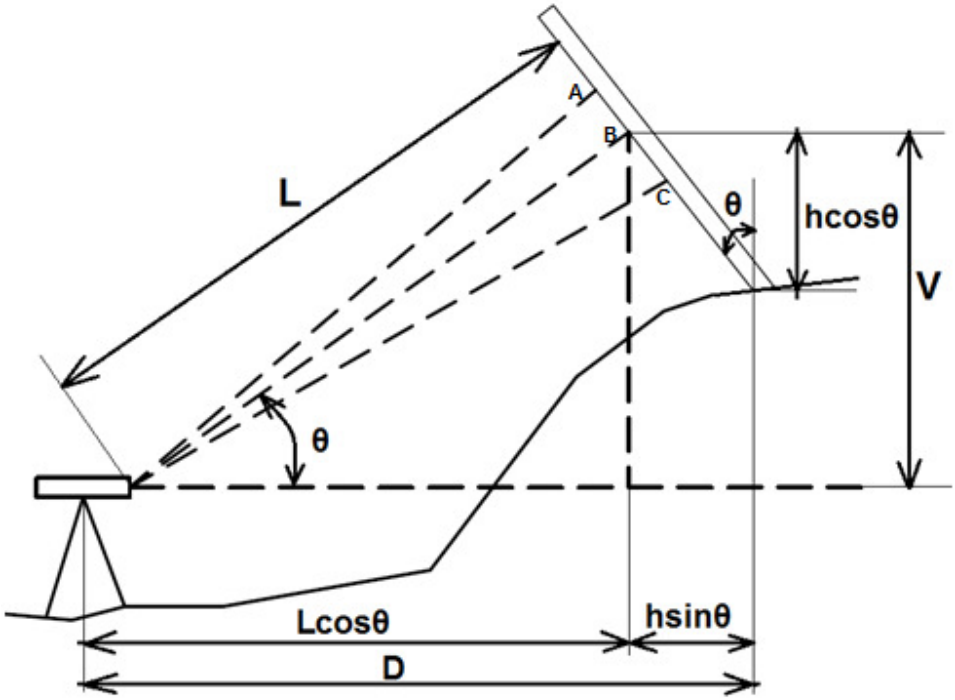
A'C' = S cos θ = projection of staff intercept on the plane perpendicular to the line of collimation

⊙ From figure, L = K (A'C') + C

=> **L = K S cos θ + C**

- D = L cos θ
- V = L sin θ = D tan θ
- RL of the given point = HI + V - h

b) <u>Staff held normal to Line of Sight:</u>



- From figure,

$$L = K S + C$$

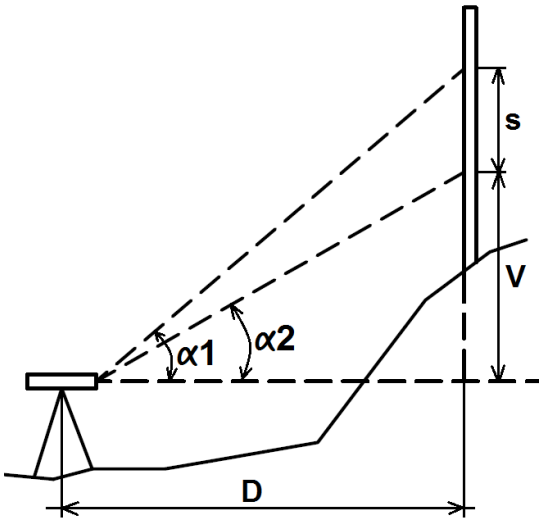
- D = L cos θ + h sin θ
- V = L sin θ
- RL of the given point = HI + V – h cos θ

TANGENTIAL TACHEOMETRY

Here, the vertical angles to the two fixed targets on the staff are observed at the instrument station. Horizontal distances to the staff station are determined in the following cases as follows.

DIFFERENT CASES INVOLVED:

1) *When both the observed angles are Angles of Elevation*



$$\text{Tan } \alpha 1 = \frac{s+V}{D} \quad \text{----------(1)}$$

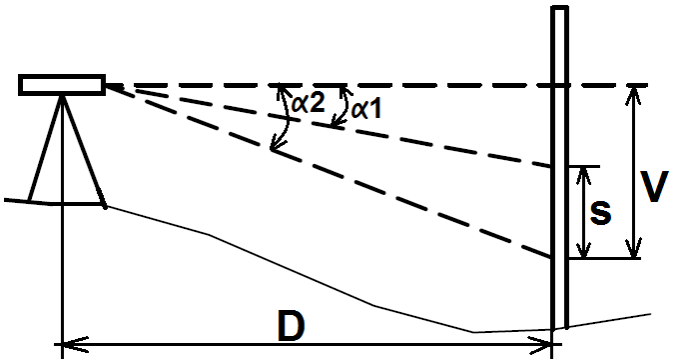
$$\text{Tan } \alpha 2 = \frac{V}{D} \quad \text{----------(2)}$$

Eliminating V from above equations:

$$\odot\ D = \frac{s}{\text{Tan } \alpha 1 - \text{Tan } \alpha 2}$$

$$\odot\ V = D \tan \alpha 2$$

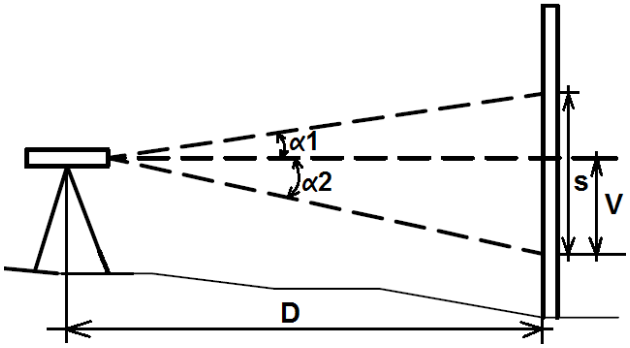
2) *When both the observed angles are Angles of Depression*



$$\odot\ D = \frac{s}{\text{Tan } \alpha 2 - \text{Tan } \alpha 1}$$

$$\odot\ V = D \tan \alpha 2$$

3) *When one angle observed is Angle of Elevation & the other an Angle of Depression*



$$\odot\ D = \frac{s}{\text{Tan } \alpha 1 + \text{Tan } \alpha 2}$$

$$\odot\ V = D \tan \alpha 2$$

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